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2.1 Organizational Charts

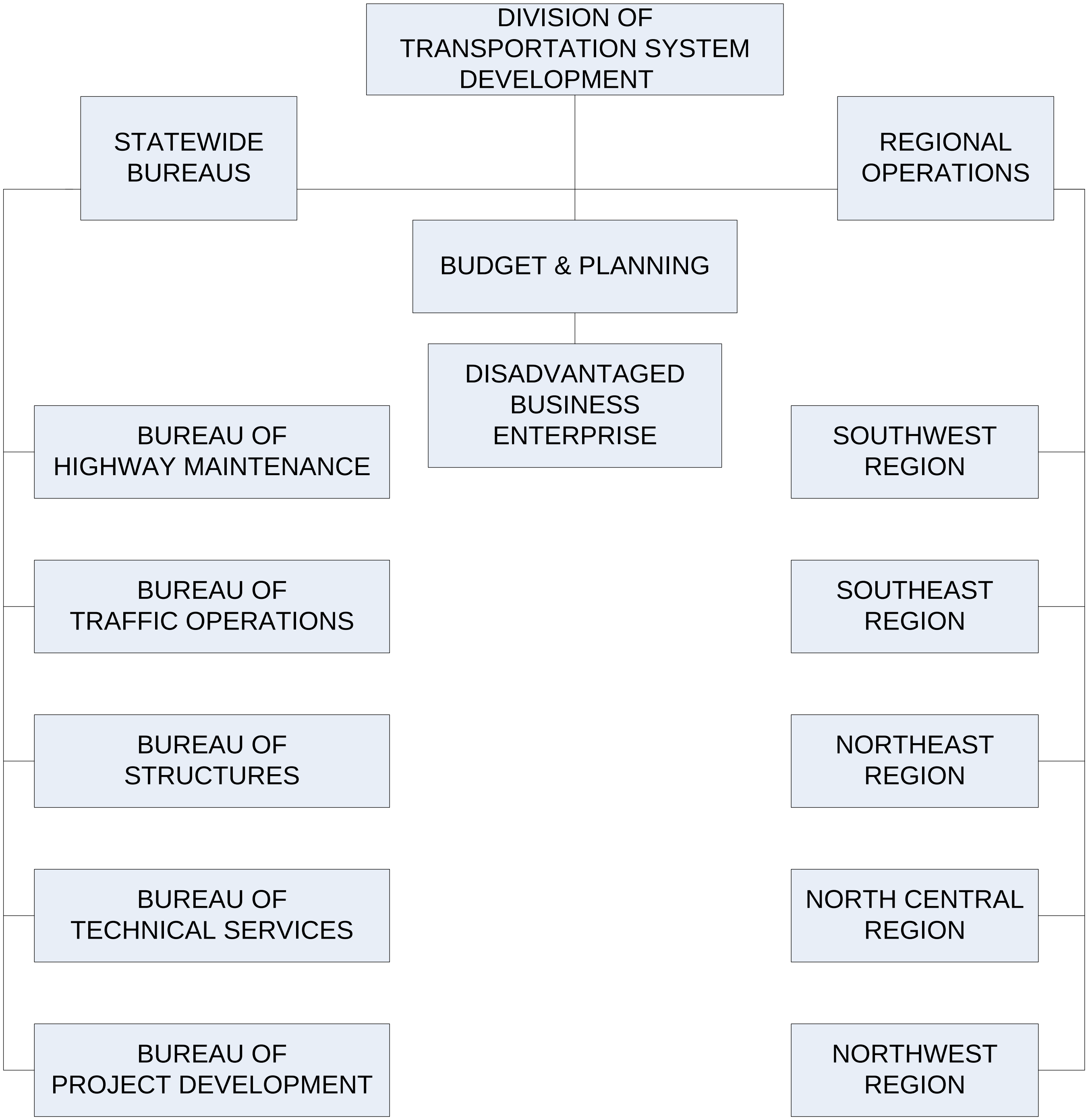


Figure 2.1-1
Division of Transportation System Development

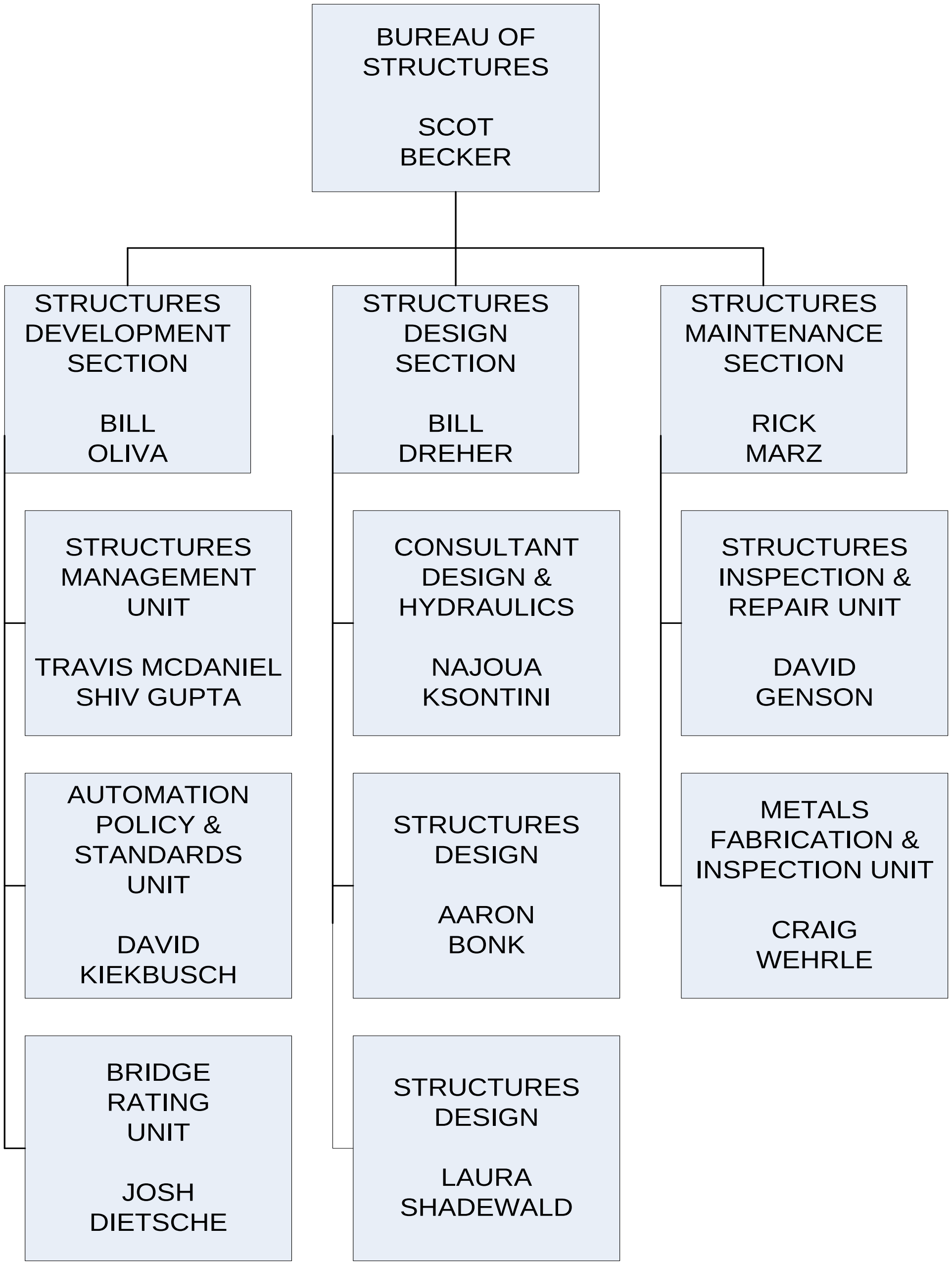
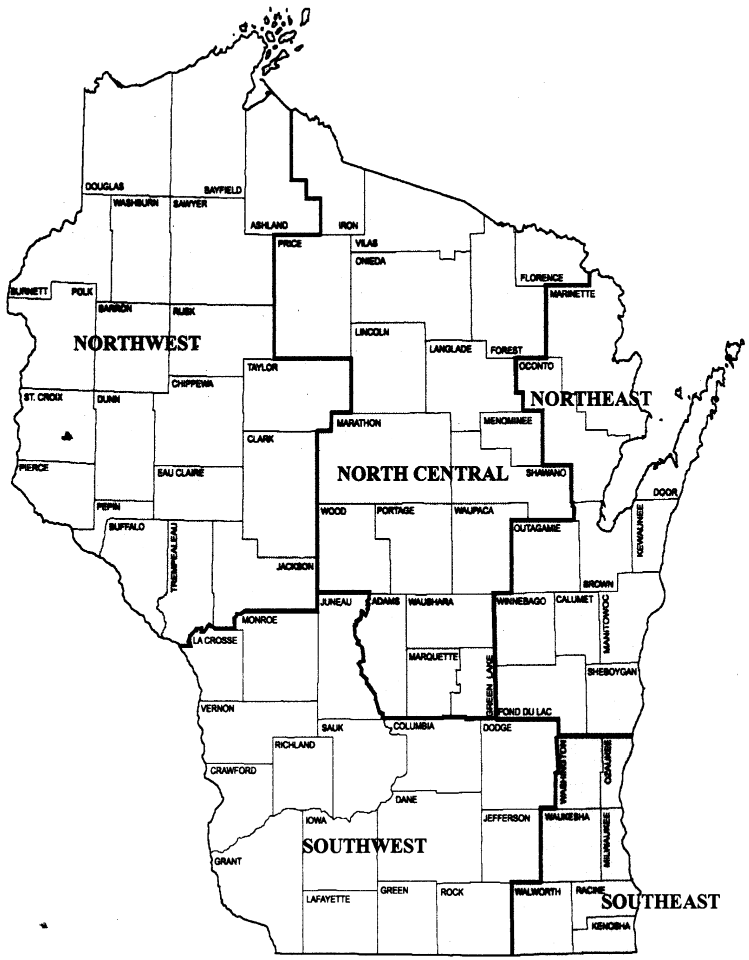


Figure 2.1-2
Bureau of Structures



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

Figure 2.1-3
Region Map

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4.1 Introduction

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

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

4.2 General Aesthetic Guidelines

Primary features – in relative order of importance:

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

Secondary features – in relative order of importance:

- Color
- Pattern and texture
- Ornamentation

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

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

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

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

Refer to the WisDOT Traffic Guidelines Manual 2-1-60 for guidance on community sensitive design signing.

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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.

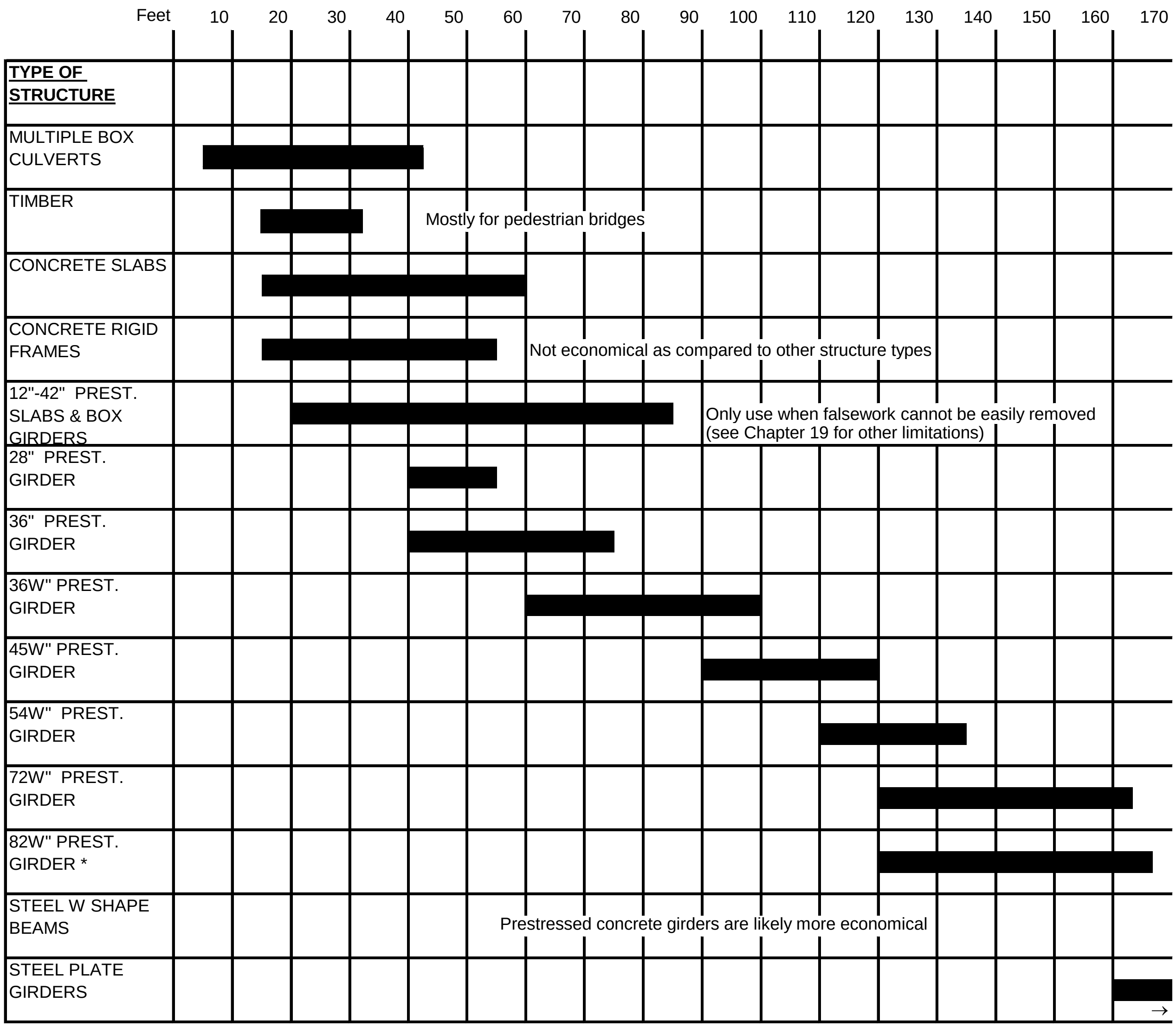
In these reports prestressed girder costs are grouped together because there is a small cost difference between girder sizes. Refer to unit costs. Concrete slab costs are also grouped together for this reason.

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
00000	Parapet Concrete Type LF & A (estimate)	LF	80.00
00000	Piling Steel Preboring	LF	122.33
00000	Preboring Cast-in-Place Concrete Piling	LF	25.00
206.6000.S	Temporary Shoring	SF	31.82
210.0100	Backfill Structure	CY	18.93
303.0115	Pit Run	CY	13.01
311.0115	Breaker Run	CY	20.20
502.0100	Concrete Masonry Bridges	CY	525.00
502.1100	Concrete Masonry Seal	CY	157.14
502.2000	Compression Joint Sealer Preformed Elastomeric (width)	LF	31.86
502.3100	Expansion Device (structure) (LS)	LF	167.60
502.3110.S	Expansion Device Modular (structure) (LS)	LF	1,346.96
502.3200	Protective Surface Treatment	SY	1.87
502.3210.S	Pigmented Protective Surface Treatment	SY	5.05
502.6500	Protective Coating Clear	GAL	86.67
503.0128	Prestressed Girder Type I 28-Inch	LF	118.63
503.0136	Prestressed Girder Type I 36-Inch	LF	157.91
503.0137	Prestressed Girders Type I 36W-Inch	LF	142.75
503.0145	Prestressed Girder Type I 45-Inch	LF	160.00
503.0146	Prestressed Girders Type I 45W-Inch	LF	157.58
503.0154	Prestressed Girder Type I 54-Inch	LF	--
503.0155	Prestressed Girder Type I 54W-Inch	LF	155.46
503.0170	Prestressed Girder Type I 70-Inch	LF	--
503.0172	Prestressed Girders Type I 72W-Inch	LF	210.78
503.0182	Prestressed Girder Type I 82W-Inch	LF	--
504.0100	Concrete Masonry Culverts	CY	445.83
504.0500	Concrete Masonry Retaining Walls	CY	549.62
505.0405	Bar Steel Reinforcement HS Bridges	LB	0.87
505.0410	Bar Steel Reinforcement HS Culverts	LB	0.86
505.0415	Bar Steel Reinforcement HS Retaining Walls	LB	0.69
505.0605	Bar Steel Reinforcement HS Coated Bridges	LB	0.91
505.0610	Bar Steel Reinforcement HS Coated Culverts	LB	0.87
505.0615	Bar Steel Reinforcement HS Coated Retaining Walls	LB	1.14
506.0105	Structural Carbon Steel	LB	3.08
506.0605	Structural Steel HS	LB	2.47
506.2605	Bearing Pads Elastomeric Non-Laminated	EACH	71.71
506.2610	Bearing Pads Elastomeric Laminated	EACH	1,248.00
506.3005	Welded Stud Shear Connectors 7/8 x 4-Inch	EACH	3.06
506.3010	Welded Stud Shear Connectors 7/8 x 5-Inch	EACH	3.15
506.3015	Welded Stud Shear Connectors 7/8 x 6-Inch	EACH	1.89
506.3020	Welded Stud Shear Connectors 7/8 x 7-Inch	EACH	3.60
506.3025	Welded Stud Shear Connectors 7/8 x 8-Inch	EACH	3.60
506.4000	Steel Diaphragms (structure)	EACH	571.20
506.5000	Bearing Assemblies Fixed (structure)	EACH	1,076.22
506.6000	Bearing Assemblies Expansion (structure)	EACH	1,072.66
507.0200	Treated Lumber and Timber	MBM	5,306.70
508.1600	Piling Treated Timber Delivered	LF	33.75

512.1000	Piling Steel Sheet Temporary	SF	10.00
513.4050	Railing Tubular Type F (structure) (LS)	LF	109.12
513.4052/4053	Railing Tubular Type F- (4 or 5) Modified (structure) (LS)	LF	137.20
513.4055	Railing Tubular Type H (structure) (LS)	LF	110.97
513.4060	Railing Tubular Type M (structure) (LS)	LF	180.26
513.4065	Railing Tubular Type PF (structure) (LS)	LF	--
513.4090	Railing Tubular Screening (structure) (LS)	LF	130.06
513.7005	Railing Steel Type C1 (structure) (LS)	LF	94.31
513.7010	Railing Steel Type C2 (structure) (LS)	LF	129.10
513.7015	Railing Steel Type C3 (structure) (LS)	LF	114.54
513.7020	Railing Steel Type C4 (structure) (LS)	LF	119.00
513.7025	Railing Steel Type C5 (structure) (LS)	LF	--
513.7030	Railing Steel Type C6 (structure) (LS)	LF	111.49
513.7050	Railing Type W (structure) (LS)	LF	118.03
514.0445	Floor Drains Type GC	EACH	1,875.00
514.2608	Downspout 8-Inch	LF	153.94
514.2625	Downspout 6-Inch	LF	130.67
516.0500	Rubberized Membrane Waterproofing	SY	25.94
517.1010.S	Concrete Staining (structure)	SF	1.18
517.1015.S	Concrete Staining Multi-Color (structure)	SF	1.90
517.1050.S	Architectural Surface Treatment (structure)	SF	3.65
550.0500	Pile Points	EACH	110.55
550.1100	Piling Steel HP 10-Inch x 42 LB	LF	32.40
550.1120	Piling Steel HP 12-Inch x 53 LB	LF	35.15
550.1125	Piling Steel HP 12-Inch x 74 LB	LF	--
550.1140	Piling Steel HP 14-Inch x 73 LB	LF	44.61
550.2102	Piling CIP Concrete 10 ¾ x 0.219-Inch	LF	--
550.2104	Piling CIP Concrete 10 ¾ x 0.25-Inch	LF	28.59
550.2106	Piling CIP Concrete 10 ¾ x 0.365-Inch	LF	35.48
550.2108	Piling CIP Concrete 10 ¾ x 0.5-Inch	LF	--
550.2122	Piling CIP Concrete 12 ¾ x 0.219-Inch	LF	--
550.2124	Piling CIP Concrete 12 ¾ x 0.25-Inch	LF	35.82
550.2126	Piling CIP Concrete 12 ¾ x 0.375-Inch	LF	42.32
550.2128	Piling CIP Concrete 12 ¾ x 0.5-Inch	LF	46.20
550.2142	Piling CIP Concrete 14 x 0.219-Inch	LF	--
550.2144	Piling CIP Concrete 14 x 0.25-Inch	LF	--
550.2146	Piling CIP Concrete 14 x 0.375-Inch	LF	--
550.2148	Piling CIP Concrete 14 x 0.5-Inch	LF	--
550.2162	Piling CIP Concrete 16 x 0.219-Inch	LF	--
550.2164	Piling CIP Concrete 16 x 0.25-Inch	LF	--
550.2166	Piling CIP Concrete 16 x 0.375-Inch	LF	--
550.2168	Piling CIP Concrete 16 x 0.5-Inch	LF	--
604.0400	Slope Paving Concrete	SY	49.20
604.0500	Slope Paving Crushed Aggregate	SY	19.20
604.0600	Slope Paving Select Crushed Material	SY	22.65
606.0100	Riprap Light	CY	48.31
606.0200	Riprap Medium	CY	45.82
606.0300	Riprap Heavy	CY	46.20
606.0500	Grouted Riprap Light	CY	--
606.0600	Grouted Riprap Medium	CY	--

606.0700	Grouted Riprap Heavy	CY	108.00
612.0106	Pipe Underdrain 6-Inch	LF	3.78
612.0206	Pipe Underdrain Unperforated 6-Inch	LF	11.82
612.0406	Pipe Underdrain Wrapped 6-Inch	LF	6.69
614.0150	Anchor Assemblies for Steel Plate Beam Guard	Each	113.28
616.0205	Fence Chain Link 5-FT	LF	--
616.0206	Fence Chain Link 6-FT	LF	10.97
616.0208	Fence Chain Link 8-FT	LF	36.99
645.0105	Geotextile Fabric Type C	SY	2.87
645.0111	Geotextile Fabric Type DF (Schedule A)	SY	3.00
645.0120	Geotextile Fabric Type HR	SY	2.92
652.0125	Conduit Rigid Metallic 2-Inch	LF	18.65
652.0135	Conduit Rigid Metallic 3-Inch	LF	29.82
652.0225	Conduit Rigid Nonmetallic Schedule 40 2-Inch	LF	4.05
652.0235	Conduit Rigid Nonmetallic Schedule 40 3-Inch	LF	5.78
657.6005.S	Anchor Assemblies Light Poles	Each	598.96
SPV.0035	HPC Masonry Superstructure	CY	501.08
SPV.0085	Stainless Steel Reinforcement	LB	3.38
SPV.0105	Parapet Concrete Type TX (LS) (estimate)	LF	184.53
SPV.0165	Anti-Graffiti Coating	SF	--
SPV.0180	Protective Polymer Coating	SY	--

Table 5.3-1

Contract Unit Bid Prices for New Structures

Item No.	Bid Item	Unit	Cost
455.0___	Asphaltic Material _________	TON	100.00
460.1___	HMA Pavement Type _____	TON	43.54
502.5002	Masonry Anchors Type L No. 4 Bars	EACH	12.24
502.5005	Masonry Anchors Type L No. 5 Bars	EACH	10.94
502.5010	Masonry Anchors Type L No. 6 Bars	EACH	15.62
502.5015	Masonry Anchors Type L No. 7 Bars	EACH	20.95
502.5020	Masonry Anchors Type L No. 8 Bars	EACH	23.38
502.5025	Masonry Anchors Type L No. 9 Bars	EACH	21.04
502.6102	Masonry Anchors Type S 1/2-Inch	EACH	25.26
502.6105	Masonry Anchors Type S 5/8-Inch	EACH	15.27
502.6110	Masonry Anchors Type S 3/4-Inch	EACH	29.64
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	19.69
505.0905	Bar Couplers No. 5	EACH	19.43
505.0906	Bar Couplers No. 6	EACH	25.75
505.0907	Bar Couplers No. 7	EACH	42.21
505.0908	Bar Couplers No. 8	EACH	50.72
505.0909	Bar Couplers No. 9	EACH	80.22
509.0301	Preparation Decks Type 1	SY	75.50
509.0302	Preparation Decks Type 2	SY	95.61
509.0500	Cleaning Decks	SY	7.53
509.1000	Joint Repair	SY	627.59
509.1200	Curb Repair	LF	48.12
509.1500	Concrete Surface Repair	SF	81.00
509.2000	Full-Depth Deck Repair	SY	486.70
509.2500	Concrete Masonry Overlay Decks	CY	411.31
509.5100.S	Polymer Overlay	SY	36.07

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 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 5.4-8
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 Structures	1	2,286	1,712,743	0	749.23

Table 5.4-9
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 5.4-10
Box Culverts

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

Table 5.4-11
Pedestrian Bridges

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

Table 5.4-12
County Timber Bridges

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

Table 5.4-13
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 5.4-14
Retaining Walls

5.4.2 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 5.4-15
Stream Crossing Structure

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 5.4-16
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 5.4-17
Box Culverts

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

Table 5.4-18
Pre-Fab Pedestrian Bridge

Pedestrian Bridges	Cost per Sq. Ft.
4	179.56

Table 5.4-19
Pedestrian Bridges

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

Table 5.4-20
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 5.4-21
Retaining Walls

5.4.3 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 5.4-22
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 5.4-23
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 5.4-24
Box Culverts

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

Table 5.4-25
Pre-Fab Pedestrian Bridge

Pedestrian Bridges	Cost per Sq. Ft.
None this Year	

Table 5.4-26
Pedestrian Bridges

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

Table 5.4-27
Railroad 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 5.4-28
Retaining Walls

5.4.4 2012 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	18	115,512	11,610,435	53.88	100.50
Reinf. Conc. Slabs (All but A5)	22	80,797	8,269,942	53.04	102.35
Reinf. Conc. Slabs (A5 Abuts)	3	6,438	739,983	53.24	114.95
Prestressed Box Girders	0				

Table 5.4-29
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	58	697,381	65,044,526	65.91	93.27
Steel Plate Girders	0				
Reinf. Conc. Slabs (All but A5)	1	5,812	491,683	43.73	84.60
Reinf. Conc. Slabs (A5 Abuts)	0				
Trapezoid Box	0				

Table 5.4-30
Grade Separation Structures

Box Culverts	No. of Culverts	Cost per Lin. Ft.
Single Cell	5	1,516.50
Twin Cell	6	3,292.00
Triple Cell	5	2,624.60
Precast	1	0

Table 5.4-31
Box Culverts

Pre-Fab Pedestrian Bridge	Cost per Sq. Ft.
B-40-761/762	325.22

Table 5.4-32
Pre-Fab Pedestrian Bridge

Pedestrian Bridges	Cost per Sq. Ft.
B-53-265	91.93

Table 5.4-33
Pedestrian Bridges

Buried Slab Bridge	Cost per Sq. Ft.
C-13-155	170.77

Table 5.4-34
Buried Slab Bridges

Retaining Walls	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Block Walls	17	30,536	1,604,280	52.54
MSE Panel Walls	25	111,365	7,215,980	64.80
Modular Walls	1	500	49,275	98.50
Concrete Walls	2	5,061	416,963	82.39
Panel Walls	2	6,476	1,094,638	169.03
Tangent Pile Walls	0			
Wired Faced MSE Wall	21	109,278	16,130,424	147.61
Secant Wall	1	12,545	2,073,665	165.30
Soldier Pile Wall	2	4,450	298,547	66.49
MSE Gravity Walls	1	975	61,470	63.05
Steel Sheet Piling	5	8,272	352,938	42.67

Table 5.4-35
Retaining Walls

5.4.5 2013 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	17	120,700	12,295,720	49.75	101.87
Reinf. Conc. Slabs (All but A5)	12	26,361	2,244,395	48.26	85.14
Reinf. Conc. Slabs (A5 Abuts)	5	8,899	992,966	49.28	111.58
Prestressed Box Girders	0				

Table 5.4-36
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	52	672,482	67,865,859	69.67	100.92
Steel Plate Girders	6	195,462	27,809,905	89.62	142.28
Reinf. Conc. Slabs (All but A5)	0				
Reinf. Conc. Slabs (A5 Abuts)	0				
Trapezoid Box	7	571,326	98,535,301	116.21	172.47

Table 5.4-37
Grade Separation Structures

Box Culverts	No. of Culverts	Cost per Lin. Ft.
Single Cell	11	1,853.00
Twin Cell	5	2,225.00
Triple Cell	0	0
Precast	3	1,079.00

Table 5.4-38
Box Culverts

Pre-Fab Pedestrian Bridge	Cost per Sq. Ft.
B-13-666	240.30
B-17-211	174.33

Table 5.4-39
Pre-Fab Pedestrian Bridge

Pedestrian Bridges	Cost per Sq. Ft.
B-13-661	222.06
B-13-656	105.60
B-13-657	106.62
B-40-784	289.02

Table 5.4-40
Pedestrian Bridges

Buried Slab Bridge	Cost per Sq. Ft.
B-24-40	182.28
B-5-403	165.57
B-13-654	210.68

Table 5.4-41
Buried Slab Bridges

Railroad Bridge	Cost per Sq. Ft.
B-40-773	1,151
B-40-774	1,541

Table 5.4-42
Railroad Bridges

Inverted T Bridge	Cost per Sq. Ft.
B-13-608	192.75
B-13-609	235.01
B-40-89	528.81

Table 5.4-43
Inverted T Bridges

Retaining Walls	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Block Walls	8	13,351	447,017	33.48
MSE Panel Walls	55	255,817	23,968,072	93.69
Modular Walls	0			
Concrete Walls	23	32,714	2,991,867	91.46
Panel Walls	7	39,495	8,028,652	203.28
Tangent Pile Walls	0			
Wired Faced MSE Wall	28	160,296	20,554,507	128.17
Secant Wall	0			
Soldier Pile Wall	0			
MSE Gravity Walls	0			
Steel Sheet Piling	0			

Table 5.4-44
Retaining Walls

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

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

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

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

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

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

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

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

6.2.2.3 Requirements of Drawing

6.2.2.3.1 Plan View

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

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

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

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

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

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

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

7. Detail the extent of slope paving or riprap.

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

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

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

11. Median width if dual highway.

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

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

14. Location and dimension of minimum vertical clearance for highway or railroad grade separation structures.

    a. The minimum vertical clearance should be noted as the "Point of Minimum Vertical Clearance" for all spans.

    b. Minimum vertical clearance is required for each span of a structure above a traveled way (i.e., roadway, railroad track, etc.).

    c. Refer to *Facilities Development Manual 11-35-1, Section 1.5* for guidance pertaining to the required locations to be checked for underclearance.

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

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

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

18. Structure numbers on plan.

19. Excavation protection for railroads.

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

21. Location of deck lighting or utilities if any.

22. Name Plate location.

23. Bench Mark Cap Location

24. Locations of surface drains on approach pavement.

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

6.2.2.3.2 Elevation View

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

1. Profile of existing groundline or streambed.

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

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

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

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

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

7. Location and dimension of minimum vertical clearance.

    a. Minimum vertical clearance is required for each span of a structure above a traveled way (i.e., roadway, railroad track, etc.).

    b. Refer to *Facilities Development Manual 11-35-1, Section 1.5* for guidance pertaining to the required locations to be checked for underclearance.

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

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

10. Location of fixed and expansion bearings.

11. Location and type of expansion devices.

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

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

6.2.2.3.3 Cross-Section View

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

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

2. Horizontal dimensions tied into a reference line or centerline of roadway.

3. Steel beam or girder spacing with beam/girder depth.

4. For prestressed girders approximate position of exterior girders.

5. Direction and amount of crown or superelevation.

6. Point referred to on profile grade.

7. Type of pier with size and number of columns proposed.

8. For solid, hammerhead or other type pier approximate size to scale.

9. If length of concrete pier cap between outer pier columns exceeds approximately 60 feet, provide an opening in the cross girder for temperature changes and concrete shrinkage, or design the pier cap for temperature and shrinkage to eliminate the opening.

10. Dimension minimum depth of bottom of footings below ditch or finished ground line or if railroad crossing below top of rail.

11. Location for public and private utilities to be carried in the superstructure. Label owner's name of utilities.

12. Location of lighting on the deck or under the deck if any.

6.2.2.3.4 Other Requirements

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

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

2. Channel change section if applicable. Approximate stream bed elevation at low point.

3. Any other view or detail which may influence the bridge type, length or clearance.

4. List design data including:

   Ultimate Stresses for Materials:

   - Concrete Superstructure
   - Concrete Substructure
   - Bar Steel Reinforcement
   - Structural Steel
   - Prestressed Concrete
   - Prestressing Steel

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

   Foundations

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

   Ratings (Plans Including Ratings that have been Changed)

   Live Load:

   Design Loading: HL-93

   Inventory Rating Factor: RF = X.XX

   Operating Rating Factor: RF = X.XX

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

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

   Ratings (Plans Including Ratings that have not been Changed)

   Live Load:

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

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

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

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

Hydraulic Data

Base Flood

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

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

- Overtopping Frequency
- Overtopping Elevation
- Overtopping Discharge

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

6.2.2.4 Utilities

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

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

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

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

6.2.3 Distribution of Exhibits

6.2.3.1 Federal Highway Administration (FHWA).

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

1. Projects on the Interstate System

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

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

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

Unusual bridges have the following characteristics:

- Difficult or unique foundation problems

- New or complex designs with unique operational or design features

- Exceptionally long spans

- Design procedures that depart from currently recognized acceptable practices

Examples of unusual bridges:

- Cable-stayed

- Suspension

- Arch

- Segmental concrete

- Movable

- Truss

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

Examples of unusual structures:

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

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

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

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

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

6.2.3.2 Coast Guard

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

6.2.3.3 Regions

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

6.2.3.4 Utilities

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

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

6.2.3.5 Other Agencies

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

6.3 Final Plans

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

6.3.1 General Requirements

6.3.1.1 Drawing Size

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

6.3.1.2 Scale

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

6.3.1.3 Line Thickness

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

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

6.3.1.4 Lettering and Dimensions

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

6.3.1.5 Notes

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

1. General Plan

2. Subsurface Exploration

3. Abutments

4. Piers

5. Superstructure and Superstructure Details

6. Railing and Parapet Details

Show all views looking up station.

6.3.2.1 General Plan (Sheet 1)

See the BOS web page, CADD Resource Files, for the latest sheet borders to be used. Sheet borders are given for new bridges, rehabilitation projects and concrete box culverts. A superstructure replacement utilizing the existing substructure, bridge widenings, as well as damaged girder replacements should use the sheet border for a new structure. See Chapter 40 - Bridge Rehabilitation for criteria as to when superstructure replacements are allowed.

1. Plan View

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

   a. Sufficient dimensions to layout structure in the field.

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

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

   On Structure Replacements

   Show existing structure in dashed-lines on Plan View.

2. Elevation View

   Same requirements as specified for preliminary plan except:

   a. Show elevation at bottom of all substructure units.

   b. Give estimated pile lengths where used.

3. Cross-Section View

   Same requirements as specified for preliminary plan except:

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

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

4. Grade Line

Same requirements as specified for preliminary plan.

5. Design and Traffic Data

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

6. Hydraulic Information, if Applicable

7. Foundations

Give soil/rock bearing capacity or pile capacity.

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

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

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

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

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

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

8. Estimated Quantities

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

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

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

9. General Notes

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

10. List of Drawings

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

11. Bench Marks

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

12. Title Block

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

13. Professional Seal

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

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

6.3.2.1.1 Plan Notes for New Bridge Construction

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

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

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

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

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

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

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

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

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

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

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

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

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

6.3.2.1.2 Plan Notes for Bridge Rehabilitation

<u>WisDOT policy item:</u>

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

It is the responsibility of the design engineer to use original structure plans, as-built structure plans, shop drawings, field surveys and structure inspection reports as appropriate when producing rehabilitation structure plans of any type (bridges, retaining walls, box culverts, sign structures, etc.). If uncertainty persists after reviewing available documentation, a field visit may be necessary by the design engineer.

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

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

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

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

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

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

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

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

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

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

6.3.2.2 Subsurface Exploration

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

1. Plan View

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

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

2. Elevation

   Show a centerline profile of existing ground elevation.

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

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

6.3.2.3 Abutments

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

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

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

1. Plan View

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

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

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

2. Elevation

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

   b. Give vertical dimension of wing.

3. Wing Elevation

4. Body Section

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

5. Wing Sections

6. Bar Steel Listing and Detail

Use the following views where necessary:

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

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

* These items are added to the packet during construction.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Initial	Underwater (UW-Probe/Visual
Routine Visual	Movable
Fracture Critical	Damage
In-Depth	Interim
Underwater (UW)-Dive	Posted
Underwater (UW)-Surv	

Table 6.3-2
Various Inspection Reports

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

6.3.5 Processing Plans

1. Before P.S. & E. Process

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

   Maintain plans as PDF on E-plan server.

2. At P.S. & E. Processing

   Prepare plans for bid letting process.

3. After Structure Construction

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

   Original plan sheets and Design Folders are discarded.

6.4 Computation of Quantities

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

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

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

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

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

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

6.4.1 Excavation for Structures Bridges (Structure)

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

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

6.4.2 Backfill Granular or Backfill Structure

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

6.4.3 Concrete Masonry Bridges

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

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

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

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

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

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

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

6.4.6 Bar Steel Reinforcement HS Stainless Bridges

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

6.4.7 Structural Steel Carbon or Structural Steel HS

See 24.2.4.

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

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

6.4.9 Piling Test Treated Timber (Structure)

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

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

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

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

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

6.4.11 Preboring CIP Concrete Piling or Steel Piling

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

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

Record the type, quantity is a Lump Sum.

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

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

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

Record this quantity to the nearest 5 cubic yards.

6.4.15 Pile Points

When recommended in soils report. Bid as each.

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

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

6.4.17 Cofferdams (Structure)

Lump Sum

6.4.18 Rubberized Membrane Waterproofing

Record the quantity to the nearest square yard.

6.4.19 Expansion Device (Structure)

Record this quantity in lump sum.

Table of Contents

9.1 General

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

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

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

- Laboratory testing of materials that are submitted or samples randomly selected.
- Inspection and/or testing at the source of fabrication or manufacture.
- Inspection and/or testing in the field by WisDOT regional personnel.
- Manufacturer's certificate of compliance and/or manufacturer's certified report of test or analysis, either as sole documentation for acceptance or as supplemental documentation.
- Inspection, evaluation and testing in the normal course of project administration of material specifications.
- Some products are on approved lists or from fabricators, manufacturers, and certified sources approved by WisDOT. Lists of approved suppliers, products, and certified sources are located at www.dot.wisconsin.gov/business/engrserv/approvedprod.htm

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

9.2 Concrete

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

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

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

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

The modulus of elasticity of concrete, E_c, is a function of the unit weight of concrete and its compressive strength **LRFD [5.4.2.4]**. For a unit weight of 0.150 kcf, the modulus of elasticity is:

$f'_c = 3.5$ ksi ; $E_c = 3600$ ksi

$f'_c = 4$ ksi ; $E_c = 3800$ ksi

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

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

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

9.3 Reinforcement Bars

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

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

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

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

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

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

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

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

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

Fabricators stock all bar sizes in 60 foot lengths. For ease of handling, the detailed length for #3 and #4 bars is limited to 45 feet. Longer bars may be used for these bar sizes at the discretion of the designer, when larger quantities are required for the structure. All other bar sizes are detailed to a length not to exceed 60 feet, except for vertical bars. Bars placed in a vertical position are limited to lengths of approximately 30 feet. The location of mandatory horizontal construction joints in pier shafts or columns will generally determine the length of vertical bars in piers. All bars are detailed to the nearest inch.

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

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

9.3.1 Development Length and Lap Splices for Deformed Bars

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

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

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

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

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

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

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

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

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

9.3.2 Bends and Hooks for Deformed Bars

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

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

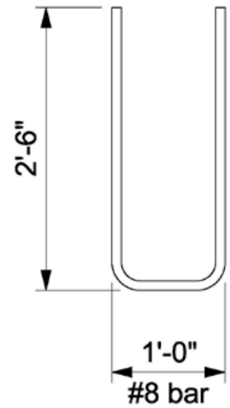


Figure 9.3-1
Bar Bend Detail (#8 bar)

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

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

9.3.3 Bill of Bars

Figure 9.9-4 shows a sample Bill of Bars table for a concrete slab. Different bar letter designations are used for abutments, slabs, and culverts, etc. If bundled bars are used, place a symbol adjacent to the bar mark of the bundled bars and a note below the Bill of Bars table stating the symbol represents bars to be bundled. A column for Bar Series is included if bars are cut.

9.3.4 Bar Series

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

The following general rules apply to the Bar Series table:

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

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

Figure 9.9-5 provides a sample layout for a Bar Series table. The Bill of Bars table will show the number of bars and the average bar length in the Series.

9.4 Steel

Structural steel is used in highway structures throughout Wisconsin. It is used for steel plate I-girders, rolled I-girders and box girders. Steel used for these three superstructure types are typically ASTM A709 Grades 36, 50 and 50W, but may also include high performance steel (HPS). Information on materials used for these superstructure types is provided in 24.2. Other types of steel superstructures are trusses, tied arches and cable-stayed bridges.

Steel is also used in other parts of the structure, such as:

- Bearings (Type A, B, A-T and top/interior plates for Laminated Elastomeric Bearings)
- Piling (H-Piles and CIP-Pile shells)
- Expansion Devices (single strip seal or modular joint)
- Drains (frame, grate and bracket)
- Railings (Type F, W, H, M, PF, Ornamental Screening, Fencing and Combination Railing)
- Steel diaphragms (attached to prestressed girders)

Structural carbon steel (ASTM A709 Grade 36) is used in components that are part of railings, laminated elastomeric bearings and for steel diaphragms attached to prestressed girders. Structural carbon steel (ASTM A36) is used in components that are part of drains. The minimum yield strength is 36 ksi.

High strength structural steel (ASTM A709 Grade 50) is used in H-piles and components that are part of railings and laminated elastomeric bearings. High strength structural weathering steel (ASTM A709 Grade 50W) is used in bearings. The minimum yield strength is 50 ksi.

Structural steel tubing (ASTM A500 Grades B,C) is used in components that are part of railings, such as posts or rail members. The minimum yield strengths will have values around 46 to 50 ksi.

Steel pipe pile material (ASTM A252 Grade 2) is used as the shell to form cast-in-place (CIP) concrete piles. The minimum yield strength is 35 ksi.

Corrugated sheet steel (AASHTO M180, Class A, Type 2) is used as rail members for steel railing Type “W”. The minimum yield strength is 50 ksi.

Stainless steel (ASTM A240 Type 304) can be found as sheets on the surface of top plates for Type A and A-T bearings. It is also used for anchor plates cast into the ends of prestressed girders.

The grade of steel, ASTM Specification (or AASHTO Material Specification) associated with the bulleted items listed above (and their components) can be found in the *Bridge Manual*

Chapters or Standards corresponding to these items. This information may also be found in the *Standard Specifications* or "*Special Provisions*".

The modulus of elasticity of steel, E_s , is 29,000 ksi and the coefficient of thermal expansion is 6.5 x 10^{-6} in/in/°F per **LRFD [6.4.1]**.

9.5 Miscellaneous Metals

The *Standard Specifications* provide the requirements for other materials made of metal that are used in highway structures. Some metals used or new products containing metal may be covered in the "*Special Provisions*".

Some of these metals, their applications and the Section of the *Standard Specifications* where they are covered are described below.

- Lubricated bronze plates are used on Type A expansion bearings. The requirements for these plates are found in Section 506.2.3.4.

- Bridge name plates are made from a casting of copper, lead, zinc and tin. The requirements for name plates are found in Section 506.2.4.

- Prestressed strands (low relaxation) are made from high tensile strength, 7-wire strands (0.5 or 0.6 inch diameter). The requirements for these strands are found in Section 503.2.3.

- Gray iron castings conforming to ASTM A48, Class 30 are used on Type GC floor drains and downspouts.

- Galvanized standard pipe conforming to ASTM A53 is used for downspouts on Type H floor drains.

- Sheet copper may be used as a waterstop for railroad bridges or as a flashing on movable bridge operator buildings. The requirements for these sheets are found in Section 506.2.3.9.

- Zinc plates may be used at deflection joints in sidewalks and parapets. The requirements for these plates are found in Section 506.2.3.10.

- Shear connectors are welded to the top flanges of steel girders to make the deck composite with the girder. Requirements for these connectors are in Section 506.2.7.

- Aluminum is used for sign bridges and some railings (Tubular Railing Type H). See Section 641.2.7 for sign bridges and Section 513.2.2 for railings that are made from aluminum. For sign bridges and sign supports made from steel, see Section 641.2.8 and 635.2 respectively.

- Steel grid floors are prefabricated grids set on girders and/or stringers. The top of the grid becomes the roadway surface. See Section 515 for the requirements for this steel.

- Welded deformed steel wire fabric has been used as an alternate to stirrup reinforcement for prestressed girders. It shall conform to the requirements of ASTM A497 as shown on the Chapter 19 – Standards.

9.6 Timber

Timber has been used for timber structures on local roads in Wisconsin. Timber has also been used for piling, railings, falsework, formwork and as backing planks between or behind piling to retain soil.

The *Bridge Manual* and the *Standard Specifications* provide requirements for timber used in highway structures. These locations are highlighted below.

- Timber structures have material requirements that are covered in Chapter 23 of the *Bridge Manual*. Requirements for the condition of the timber members and applicable preservative treatments are covered in Section 507 of the *Standard Specifications*.

- Timber railings for timber structures have material requirements that are covered in Chapter 23 of the *Bridge Manual*. Requirements for the condition of the timber members are covered in Section 507 of the *Standard Specifications*.

- Timber backing plank requirements are covered in 12.10.

9.7 Miscellaneous Materials

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

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

Other materials or products used on highway structures are:

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

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

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

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

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

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

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

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

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

9.8 Painting

All highway grade separation structures require steel girders to be painted because unpainted steel is subject to additional corrosion from vehicle salt spray. Additional discussion on painting is presented in Chapter 24 – Steel Girder Structures. The current paint system used is the three coat epoxy system specified in Section 517 of the *Standard Specifications*.

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

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

Table 9.8-1
Standard Colors for Steel Girders

[1] Shop applied color for weathering steel.

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

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

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

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

The primary reason for painting steel structures is for the protection of the steel surface. Appearance is a secondary function that is maintained by using compatible top coatings over epoxy systems.

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

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

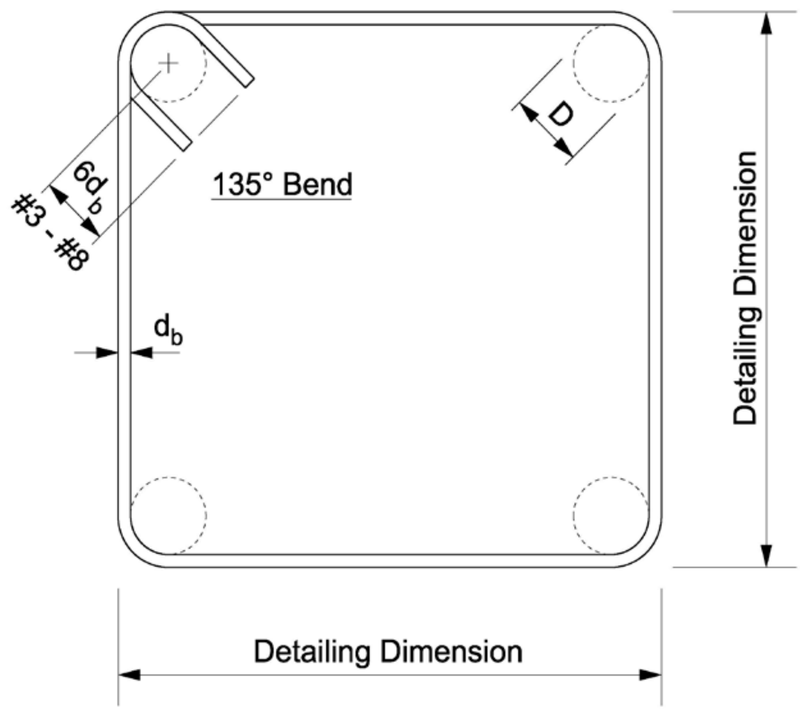
Refer to Section 1.3.14 of the *Wisconsin Structure Inspection Manual* for the criteria covering spot painting versus complete painting of existing structures. This Section provides information for evaluating the condition of a paint system and recommended maintenance.

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

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

Table 9.8-2
Standard Colors for Concrete

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



Stirrup Bar Length equals sum of all Detailing Dimensions plus “Stirrup Add-On” from table

d_b = nominal diameter of reinforcing bar (in)

Definition of Standard Hooks **LRFD [5.10.2.1]**

MINIMUM BEND DIAMETER (D) – **LRFD [5.10.2.3]**

D = 4d_b FOR #3 THRU #5

D = 6d_b FOR #6 THRU #8

BAR SIZE	D	STIRRUP ADD-ON
3	1 ½	5
4	2	6
5	2 ½	8
6	4 ½	10
7	5 ¼	12
8	6	13

Figure 9.9-3
Standard Details and Bends for Deformed Transverse Reinforcement
(Closed Stirrups)

BILL OF BARS

NOTE: THE FIRST OR FIRST TWO DIGITS OF THE BAR MARK SIGNIFIES THE BAR SIZE.

BAR MARK	COAT	NO. REQ'D	LENGTH	BENT	BAR SERIES	LOCATION
S501		10	4-2		∆	SLAB - TRANS.
S502		20	6-3		∆	SLAB - TRANS.
S503	X	19	42-8			SLAB - LONG.

∆ LENGTH SHOWN FOR BAR IS AN AVERAGE LENGTH AND SHOULD ONLY BE USED FOR BAR WEIGHT CALCULATIONS. SEE BAR SERIES TABLE FOR ACTUAL LENGTHS.

Figure 9.9-4
Bill of Bars

BAR SERIES TABLE

MARK	NO. REQ'D.	LENGTH
S501	1 SERIES OF 10	2-1 TO 6-3
S502	2 SERIES OF 10	3-2 TO 9-5

BUNDLE AND TAG EACH SERIES SEPARATELY

Figure 9.9-5
Bar Series Table

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

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

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

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

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Table of Contents

12.1 General

Abutments are used at the ends of bridges to retain the embankment and to carry the vertical and horizontal loads from the superstructure to the foundation, as illustrated in Figure 12.1-1. The design requirements for abutments are similar to those for retaining walls and for piers; each must be stable against overturning and sliding. Abutment foundations must also be designed to prevent differential settlement and excessive lateral movements.

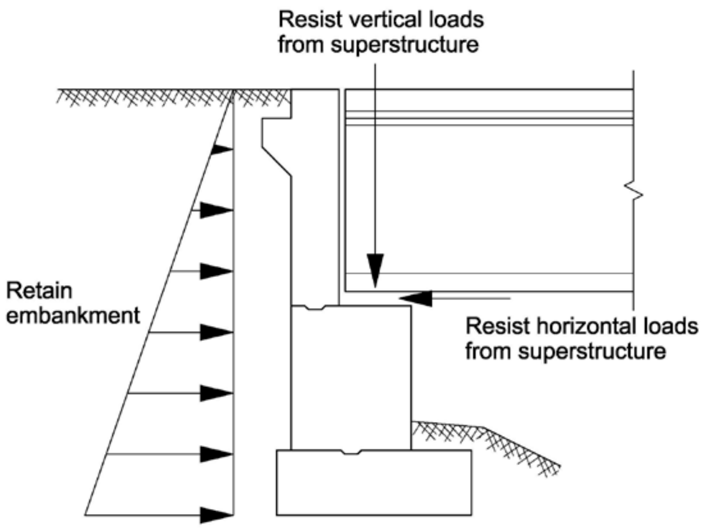


Figure 12.1-1
Primary Functions of an Abutment

The components of a typical abutment are illustrated in Figure 12.1-2.

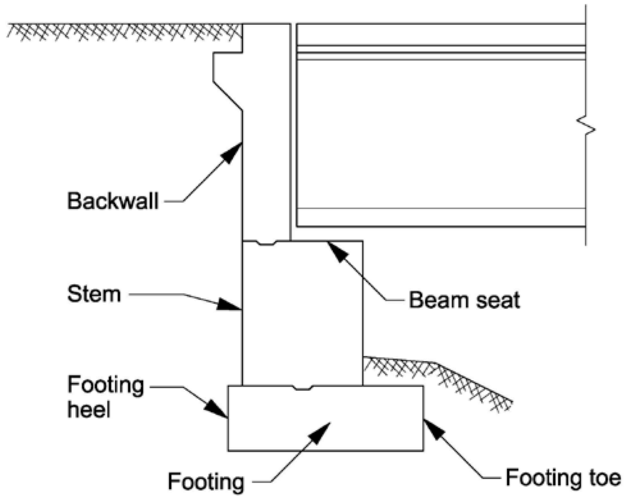


Figure 12.1-2
Components of an Abutment

Many types of abutments can be satisfactorily utilized for a particular bridge site. Economics is usually the primary factor in selecting the type of abutment to be used. For river or stream crossings, the minimum required channel area and section are considered. For highway overpasses, minimum horizontal clearances and sight-distances must be maintained.

An abutment built on a slope or on top of a slope is less likely to become a collision obstacle than one on the bottom of the slope and is more desirable from a safety standpoint. Aesthetics is also a factor when selecting the most suitable abutment type.

12.2 Abutment Types

Several different abutment types can be used, including full-retaining, semi-retaining, sill, spill-through or open, pile-encased and special designs. Each of these abutment types is described in the following sections.

12.2.1 Full-Retaining

A full-retaining abutment is built at the bottom of the embankment and must retain the entire roadway embankment, as shown in Figure 12.2-1. This abutment type is generally the most costly. However, by reducing the span length and superstructure cost, the total structure cost may be reduced in some cases. Full-retaining abutments may be desirable where right of way is critical.

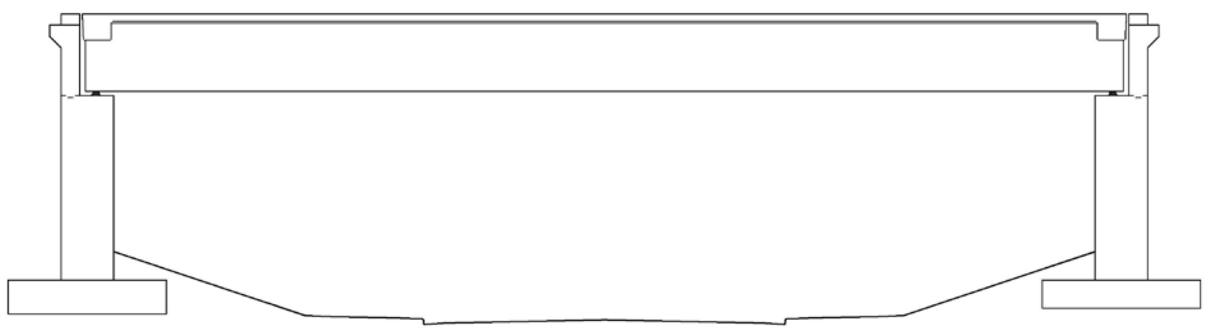


Figure 12.2-1
Full-Retaining Abutment

Rigid-frame structures use a full-retaining abutment poured monolithically with the superstructure. If both abutments are connected by fixed bearings to the superstructure (as in rigid frames), the abutment wings are joined to the body by a mortised expansion joint. For a non-skewed abutment, this enables the body to rotate about its base and allows for superstructure contraction and expansion due to temperature and shrinkage, assuming that rotation is possible.

An objectionable feature of full-retaining abutments is the difficulty associated with placing and compacting material against the body and between the wing walls. It is possible that full-retaining abutments may be pushed out of vertical alignment if heavy equipment is permitted to work near the walls, and this temporary condition is not accounted for in a temporary load combination. The placement of the embankment after abutment construction may cause foundation settlement. For these reasons, as much of the roadway embankment as practical should be in place before starting abutment construction. Backfilling above the beam seat is prohibited until the superstructure is in place.

Other disadvantages of full-retaining abutments are:

- Minimum horizontal clearance

- Minimum sight distance when roadway underneath is on a curved alignment
- Collision hazard when abutment front face is not protected
- Settlement

12.2.2 Semi-Retaining

The semi-retaining abutment (Types A3 and A4) is built somewhere between the bottom and top of the roadway embankment, as illustrated in Figure 12.2-2. It provides more horizontal clearance and sight distance than a full-retaining abutment. Located on the embankment slope, it becomes less of a collision hazard for a vehicle that is out of control.

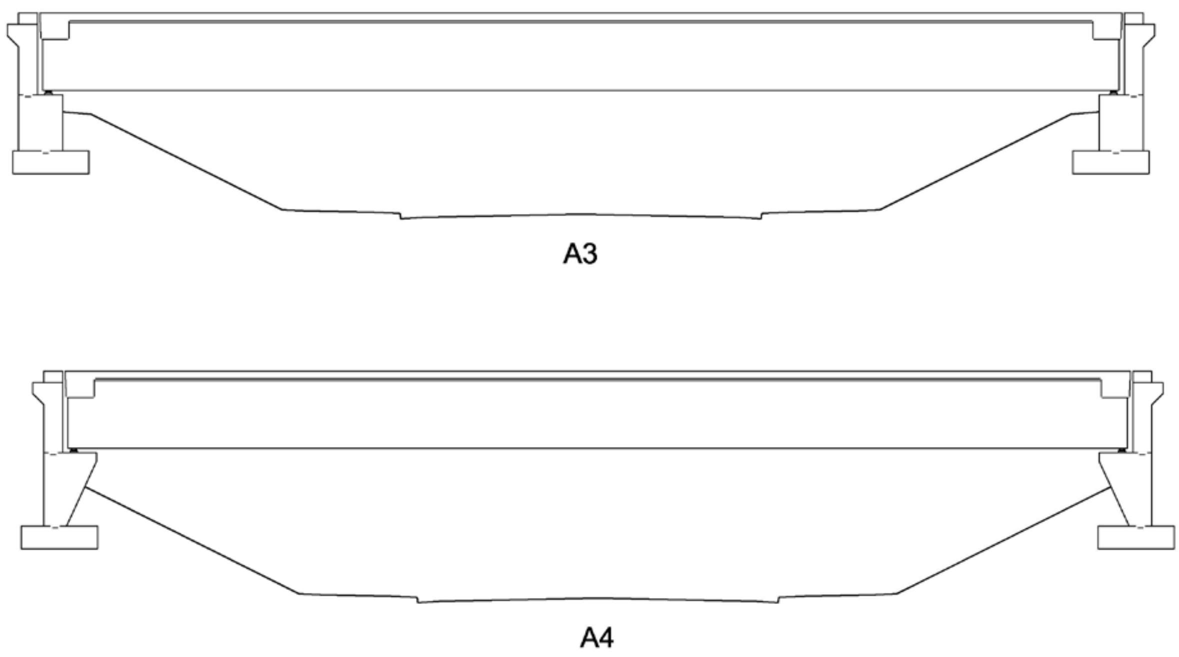
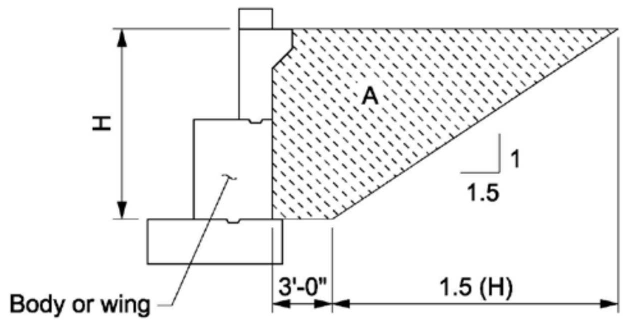


Figure 12.2-2
Semi-Retaining Abutment

The description of full-retaining abutments in 12.2.1 generally applies to semi-retaining abutments as well. They are used primarily in highway-highway crossings as a substitute for a shoulder pier and sill abutment. Semi-retaining abutments generally are designed with a fixed base, allowing wing walls to be rigidly attached to the abutment body. The wings and the body of the abutment are usually poured monolithically.

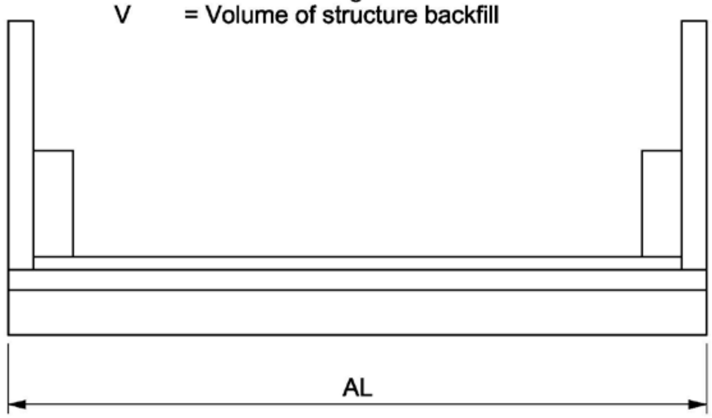
For deep girder bridges (girder height > 60 inches) the aesthetic appearance of the A4 abutment is minimized and the A3 abutment should be considered.



WING ELEVATION

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

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



ABUTMENT PLAN

STRUCTURE BACKFILL

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

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

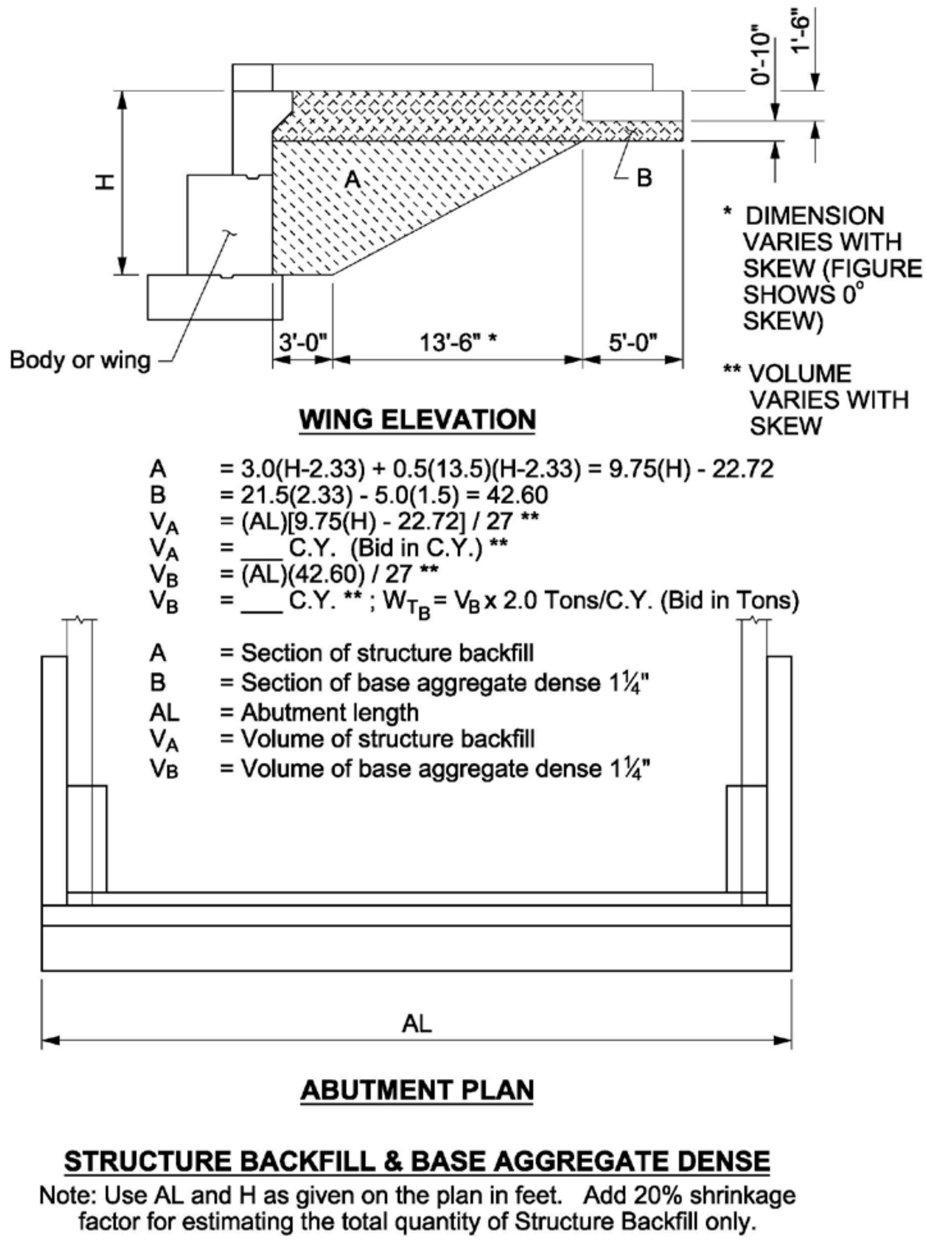


Figure 12.6-2

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

12.7 Selection of Standard Abutment Types

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

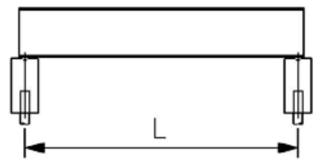
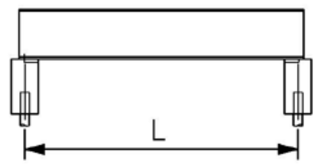
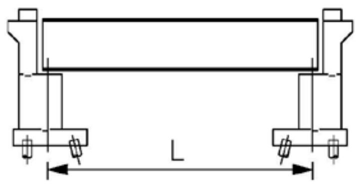
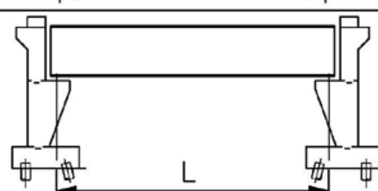
Abutment Arrangements			Superstructures		
			S=Skew, AL=Abutment Length		
L=Length of continuous superstructure between abutments			Concrete Slab Spans	Prestressed Girders	Steel Girders
(1) Type A1 with fixed seat		Type A1 with fixed seat	a. $L \le 150'$ $S \le 30°$ $AL \le 50'$	a. $L \le 150'$ $S \le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

Figure 12.7-1
Recommended Guide for Abutment Type Selection

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

12.9.2 Beam Seats

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

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

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

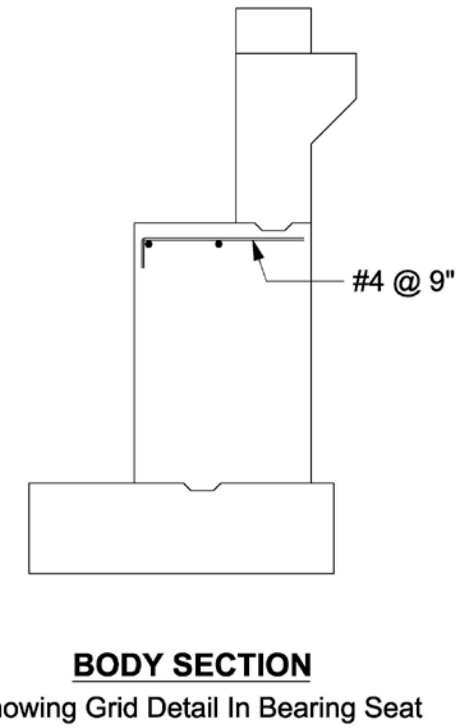


Figure 12.9-1
Reinforcing Grid Detail in Bearing Seat

12.10 Timber Abutments

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

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

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

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

12.11 Bridge Approach Design and Construction Practices

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

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

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

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

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

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

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

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

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

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

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

13.10 Pile Bent and Pile Encased Pier Analysis

WisDOT policy item:

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

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

WisDOT policy item:

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

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

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

Where:

F = Total factored vertical load (kips)

n = Number of piles

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

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

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

In which:

d = Distance of pile from pile group centroid

c = Distance from outermost pile to pile group centroid

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

13.15 Design Examples

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

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

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

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

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

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

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

The slenderness ratio for each axis now follows:

$$K_x := 2.1$$

$$K_y := 1.0$$

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

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

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

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

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

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

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

$$\beta_d := 0$$

$$E_c := 33000 \cdot w_c^{1.5} \cdot \sqrt{f'_c} \qquad \boxed{E_c = 3587} \text{ ksi}$$

$$\boxed{E_s = 29000.00} \text{ ksi}$$

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

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

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

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

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

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

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

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

The final parameter necessary for the calculation of the amplification factor is the phi-factor for compression. This value is defined as follows:

$$\phi_{axial} := 0.75$$

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

$As_{col} := 2.53$ in² per foot, based on #10 bars at 6-inch spacing

$b := 12$ inches

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

$\beta_1 := 0.85$

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

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

$\varepsilon_c := 0.002$ Upper strain limit for compression controlled sections, f_y = 60 ksi

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

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

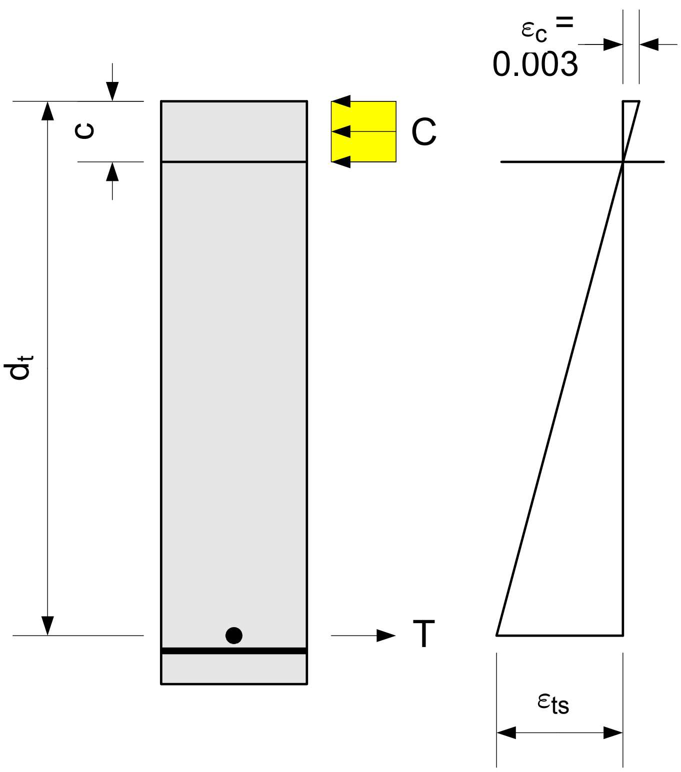


Figure E13-1.9-2
Strain Limit Tension Control Check

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

Therefore, the section is tension controlled and phi shall be equal to 0.9.

$\phi_t := 0.9$

The longitudinal moment magnification factor will now be calculated as follows:

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

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

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

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

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

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

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

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

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

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

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

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

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

$R_{xx} := \frac{Pu_{Mom\_xx}}{L_{ftg\_xx}}$ $\boxed{R_{xx} = 57.43}$ kips per foot

Estimation of applied factored load per foot in the "Y" direction:

$Pu_{Mom\_yy} := Pu \cdot 4$ $\boxed{Pu_{Mom\_yy} = 1056.79}$ kips

$R_{yy} := \frac{Pu_{Mom\_yy}}{L_{ftg\_yy}}$ $\boxed{R_{yy} = 88.07}$ kips per foot

$arm_{xx} := 2.5$ feet

$arm_{yy} := 2.25$ feet

The moment on a per foot basis is then:

$Mu_{xx} := R_{xx} \cdot arm_{xx}$ $\boxed{Mu_{xx} = 143.59}$ kip-ft per foot

$Mu_{yy} := R_{yy} \cdot arm_{yy}$ $\boxed{Mu_{yy} = 198.15}$ kip-ft per foot

Once the maximum moment at the critical section is known, flexure reinforcement must be determined. The footing flexure reinforcement is located in the bottom of the footing and rests on top of the piles.

Assume #8 bars:

$bar\_diam8 := 1.0$ inches

$bar\_area8 := 0.79$ in^2

$\boxed{f_y = 60}$ ksi

The footing minimum tensile reinforcement requirements will be calculated. The tensile reinforcement provided must be enough to develop a factored flexural resistance at least equal to the lesser of the cracking strength or 1.33 times the factored moment from the applicable strength load combinations, **LRFD [5.7.3.3.2]**.

The cracking strength is calculated as follows, **LRFD[5.7.3.3.2]**:

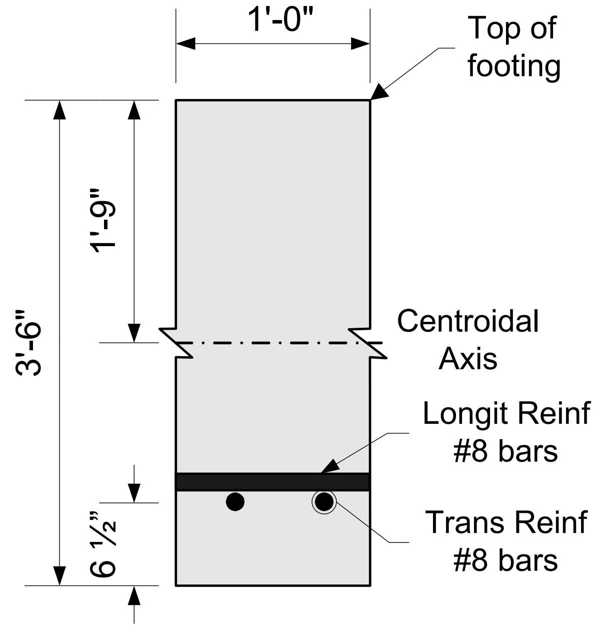


Figure E13-1.11-1
Footing Cracking Moment Dimensions

$f_r := 0.24 \cdot \sqrt{f'_c}$ $\boxed{f_r = 0.45}$ ksi

$S_g := \dfrac{b\left(H_{ftg} \cdot 12\right)^2}{6}$ $\boxed{S_g = 3528}$ in⁴

$y_t := \dfrac{H_{ftg} \cdot 12}{2}$ $\boxed{y_t = 21}$ in

$M_{cr} = \gamma_3(\gamma_1 \cdot f_r)S_g$ therefore, $M_{cr} = 1.1(f_r)S_g$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement for A615, Grade 60 reinforcement

$M_{cr} := 1.1 f_r \cdot S_g \cdot \dfrac{1}{12}$ $\boxed{M_{cr} = 145.21}$ kip-ft

1.33 times the factored controlling footing moment is:

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

$\boxed{1.33 \cdot Mu_{ftg} = 263.54}$ kip-ft

$M_{Design} := \min(M_{cr}, 1.33 \cdot Mu_{ftg})$ $\boxed{M_{Design} = 145.21}$ kip-ft

Mu_{ftg} exceeds M_{Design}, therefore set $M_{Design} = Mu_{ftg}$

Since the transverse moment controlled, M_{yy}, detail the transverse reinforcing to be located directly on top of the piles.

Effective depth, d_e = total footing thickness - cover - 1/2 bar diameter

$d_e := H_{ftg} \cdot 12 - Cover_{fb} - \frac{bar\_diam8}{2}$ $\boxed{d_e = 35.5}$ in

Solve for the required amount of reinforcing steel, as follows:

$\phi_f := 0.90$

$\boxed{b = 12}$ in

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

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

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

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

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

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

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

$\boxed{check = "OK"}$

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

E13-1.11.2 Punching Shear Check

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

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

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

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

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

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

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

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

$\boxed{b = 12}$ in

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

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

Effective depth for each axis:

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

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

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

E13-2.6 Pier Cap Design

Calculate positive and negative moment requirements.

E13-2.6.1 Positive Moment Capacity Between Columns

It is assumed that there will be two layers of positive moment reinforcement. Therefore the effective depth of the section at the pier is:

$cover := 2.5$ in

In accordance with **LRFD [5.10.3.1.3]** the minimum clear space between the bars in layers is one inch or the nominal diameter of the bars.

$spa_{clear} := 1.75$ in

$bar_{stirrup} := 5$ (transverse bar size)

$Bar_D(bar_{stirrup}) = 0.63$ in (transverse bar diameter)

$Bar_{No\_pos} := 9$

$Bar_D(Bar_{No\_pos}) = 1.13$ in (Assumed bar size)

$$d_e := cap_H \cdot 12 - cover - Bar_D(bar_{stirrup}) - Bar_D(Bar_{No\_pos}) - \frac{spa_{clear}}{2}$$

$d_e = 42.87$ in

For flexure in non-prestressed concrete, $\phi_f := 0.9$.

The width of the cap:

$b_w := cap_W \cdot 12$ $b_w = 42$ in

$Mu_{pos} = 2372$ kip-ft

$$R_u := \frac{Mu_{pos} \cdot 12}{\phi_f \cdot b_w \cdot d_e^{\,2}}$$ $R_u = 0.4097$ ksi

$$\rho := 0.85 \frac{f'_c}{f_y} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot R_u}{0.85 \cdot f'_c}}\right)$$ $\rho = 0.00738$

$A_s := \rho \cdot b_w \cdot d_e$ $A_s = 13.28$ in²

This requires $n_{bars\_pos} := 14$ bars. Use $n_{bars\_pos1} := 9$ bars in the bottom layer and $n_{bars\_pos2} := 5$ bars in the top layer. Check spacing requirements.

$$spa_{pos} := \frac{b_w - 2 \cdot (cover + Bar_D(bar_{stirrup})) - Bar_D(Bar_{No\_pos})}{n_{bars\_pos1} - 1}$$

$spa_{pos} = 4.33$ in

$clear_{spa} := spa_{pos} - Bar_D(Bar_{No\_pos})$ $\boxed{clear_{spa} = 3.2}$ in

The minimum clear spacing is equal to 1.5 times the maximum aggregate size of 1.5 inches.

$spa_{min} := 1.5 \cdot 1.5$ $\boxed{spa_{min} = 2.25}$ in

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

$As_{prov\_pos} := Bar_A(Bar_{No\_pos}) \cdot n_{bars\_pos}$ $\boxed{As_{prov\_pos} = 14}$ in^2

$a := \dfrac{As_{prov\_pos} \cdot f_y}{0.85 \cdot b_w \cdot f'_c}$ $\boxed{a = 6.72}$ in

$Mn_{pos} := As_{prov\_pos} \cdot f_y \cdot \left(d_e - \dfrac{a}{2}\right) \cdot \dfrac{1}{12}$ $\boxed{Mn_{pos} = 2766}$ kip-ft

$Mr_{pos} := \phi_f \cdot Mn_{pos}$ $\boxed{Mr_{pos} = 2489}$ kip-ft

$\boxed{Mu_{pos} = 2372}$ kip-ft

Is M_u less than M_r? $\boxed{check = \text{"OK"}}$

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

$S_{cap} := \dfrac{(cap_W \cdot 12) \cdot (cap_H \cdot 12)^2}{6}$ $\boxed{S_{cap} = 16128}$ in^3

$f_r := 0.24 \cdot \sqrt{f'_c}$ $\boxed{f_r = 0.45}$ ksi

$M_{cr} = \gamma_3(\gamma_1 \cdot f_r)S_{cap}$ therefore, $M_{cr} = 1.1(f_r)S_{cap}$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement for A615, Grade 60 reinforcement

$M_{cr} := 1.1 \cdot f_r \cdot S_{cap} \cdot \dfrac{1}{12}$ $\boxed{M_{cr} = 664}$ kip-ft

$\boxed{1.33 \cdot Mu_{pos} = 3155}$ kip-ft

Is M_r greater than the lesser value of M_{cr} and 1.33*M_u? $\boxed{check = \text{"OK"}}$

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

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

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

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

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

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

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

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

The height of the section, h, is:

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

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

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

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

$\boxed{spa_{pos} = 4.33}$ in

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

E13-2.6.2 Positive Moment Reinforcement Cut Off Location

Terminate the top row of bars where bottom row of reinforcement satisfies the moment diagram

$spa' := spa_{pos}$ $\boxed{spa' = 4.33}$ in

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

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

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

$\boxed{d_{e'} = 44.31}$ in

$$M_{n'} := As' \cdot f_y \cdot \left(d_{e'} - \frac{a'}{2}\right) \cdot \frac{1}{12}$$ $\boxed{M_{n'} = 1897}$ kip-ft

$$M_{r'} := \phi_f \cdot M_{n'}$$ $\boxed{M_{r'} = 1707}$ kip-ft

Based on the moment diagram, try locating the first cut off at $cut_{pos} := 10.7$ feet from the CL of the left column. Note that the Service I crack control requirements control the location of the cut off.

$\boxed{M_{r'} = 1707}$ kip-ft

$\boxed{Mu_{cut1} = 1538}$ kip-ft

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

Is Mu_{cut1} less than M_r? $\boxed{check = \text{"OK"}}$

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

$\boxed{M_{cr} = 664}$ kip-ft

$\boxed{1.33 \cdot Mu_{cut1} = 2045}$ kip-ft

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

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

$$\rho' := \frac{As'}{b_w \cdot d_{e'}}$$ $\boxed{\rho' = 0.00484}$

$$k' := \sqrt{(\rho' \cdot n)^2 + 2 \cdot \rho' \cdot n} - \rho' \cdot n$$ $\boxed{k' = 0.24}$

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

$Ms_{cut1} = 1051$ kip-ft

$f_{s'} := \frac{Ms_{cut1}}{As' \cdot j' \cdot d_{e'}} \cdot 12 \quad \leq 0.6\ f_y$ $\quad f_{s'} = 34.39$ ksi $\leq 0.6\ f_y$ O.K.

$\beta = 1.12$

$\gamma_e = 1$

$S_{max'} := \frac{700\gamma_e}{\beta \cdot f_{s'}} - 2 \cdot d_c$ $\quad S_{max'} = 10.81$ in

$spa' = 4.33$ in

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

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

$d_{e'} = 44.31$ in

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

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

$BarExtend_{pos} = 44.31$ in

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

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

$cut_{pos} + \frac{BarExtend_{pos}}{12} = 14.39$

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

Similar calculations show that the second layer bottom mat bars can also be terminated at a distance of 2.0 feet from the CL of the left column. At least one quarter of the bars shall be extended past the centerline of the support for continuous spans. Therefore, run the bottom layer bars to the end of the cap.

E13-2.6.3 Negative Moment Capacity at Face of Column

It is assumed that there will be one layer of negative moment reinforcement. Therefore the effective depth of the section at the pier is:

$\boxed{cover = 2.5}$ in

$\boxed{bar_{stirrup} = 5}$ (transverse bar size)

$\boxed{Bar_D(bar_{stirrup}) = 0.63}$ in (transverse bar diameter)

$Bar_{No\_neg} := 8$

$\boxed{Bar_D(Bar_{No\_neg}) = 1.00}$ in (Assumed bar size)

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

$\boxed{d_{e\_neg} = 44.38}$ in

For flexure in non-prestressed concrete, $\boxed{\phi_f = 0.9}$.

The width of the cap:

$\boxed{b_w = 42}$ in

$\boxed{Mu_{neg} = -1174}$ kip-ft

$$R_{u\_neg} := \frac{|Mu_{neg}| \cdot 12}{\phi_f \cdot b_w \cdot d_{e\_neg}^{\,2}}$$

$\boxed{R_{u\_neg} = 0.1892}$ ksi

$$\rho_{neg} := 0.85 \frac{f'_c}{f_y} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot R_{u\_neg}}{0.85 \cdot f'_c}}\right)$$

$\boxed{\rho_{neg} = 0.00326}$

$$A_{s\_neg} := \rho_{neg} \cdot b_w \cdot d_{e\_neg}$$

$\boxed{A_{s\_neg} = 6.08}$ in²

This requires $n_{bars\_neg} := 9$ bars. Check spacing requirements.

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

$\boxed{spa_{neg} = 4.34}$ in

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

$\boxed{clear_{spa\_neg} = 3.34}$ in

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

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

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

$Mn_{neg} := As_{prov\_neg} \cdot f_y \cdot \left(d_{e\_neg} - \dfrac{a_{neg}}{2}\right) \cdot \dfrac{1}{12}$ $\boxed{Mn_{neg} = 1508}$ kip-ft

$Mr_{neg} := \phi_f \cdot Mn_{neg}$ $\boxed{Mr_{neg} = 1358}$ kip-ft

$\boxed{Mu_{neg} = 1174}$ kip-ft

Is M_u less than M_r? $\boxed{check = "OK"}$

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

$\boxed{M_{cr} = 664}$ kip-ft

$\boxed{1.33 \cdot Mu_{neg} = 1561}$ kip-ft

Is M_r greater than the lesser value of M_{cr} and 1.33*M_u? $\boxed{check = "OK"}$

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

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

$\boxed{n = 8}$

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

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

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

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

$f_{s\_neg} := \dfrac{Ms_{neg}}{As_{prov\_neg} \cdot j_{neg} \cdot d_{e\_neg}} \cdot 12 \quad \leq 0.6\ f_y$ $f_{s\_neg} = 34.8$ ksi $\leq 0.6\ f_y$ O.K.

The height of the section, h, is: $\boxed{h = 48}$ in

$\beta_{neg} := 1 + \dfrac{d_{c\_neg}}{0.7 \cdot (h - d_{c\_neg})}$ $\boxed{\beta_{neg} = 1.12}$

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

$S_{max\_neg} := \dfrac{700\gamma_e}{\beta_{neg} \cdot f_{s\_neg}} - 2 \cdot d_{c\_neg}$ $\boxed{S_{max\_neg} = 10.76}$ in

$\boxed{spa_{neg} = 4.34}$ in

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

E13-2.6.4 Negative Moment Reinforcement Cut Off Location

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

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

$spa'_{neg} := spa_{neg} \cdot 2$ $\boxed{spa'_{neg} = 8.69}$ in

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

$a'_{neg} := \dfrac{As'_{neg} \cdot f_y}{0.85 \cdot b_w \cdot f'_c}$ $\boxed{a'_{neg} = 1.89}$ in

$\boxed{d_{e\_neg} = 44.38}$ in

$M_{n'\_neg} := As'_{neg} \cdot f_y \cdot \left(d_{e\_neg} - \dfrac{a'_{neg}}{2}\right) \cdot \dfrac{1}{12}$ $\boxed{M_{n'\_neg} = 853}$ kip-ft

$M_{r'\_neg} := \phi_f \cdot M_{n'\_neg}$ $\boxed{M_{r'\_neg} = 768}$ kip-ft

Based on the moment diagram, try locating the cut off at $cut_{neg} := 15.3$ feet from the CL of the left column. Note that the Service I crack control requirements control the location of the cut off.

$M_{r'\_neg} = 768$ kip-ft

$Mu_{neg\_cut} = 577$ kip-ft

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

Is Mu_{cut1} less than $M_{r'}$? check = "OK"

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

$M_{cr} = 664$ kip-ft

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

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

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

$$\rho'_{neg} := \frac{As'_{neg}}{b_w \cdot d_{e\_neg}}$$ $\rho'_{neg} = 0.00211$

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

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

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

$$f_{s'\_neg} := \frac{Ms_{neg\_cut}}{As'_{neg} \cdot j'_{neg} \cdot d_{e\_neg}} \cdot 12 \quad \le 0.6\, f_y$$ $f_{s'\_neg} = 27.79$ ksi $\le 0.6\, f_y$ O.K.

$\beta_{neg} = 1.12$

$\gamma_e = 1$

$$S_{max'\_neg} := \frac{700 \gamma_e}{\beta_{neg} \cdot f_{s'\_neg}} - 2 \cdot d_{c\_neg}$$ $S_{max'\_neg} = 15.30$ in

$spa'_{neg} = 8.69$ in

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

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

$d_{e\_neg} = 44.38$ in

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

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

$BarExtend_{neg} = 44.38$ in

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

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

The cut off location is determined by the following:

$cut_{neg} - \frac{BarExtend_{neg}}{12} = 11.6$ ft

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

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

$cutoff_{location} = 11.6$ ft

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

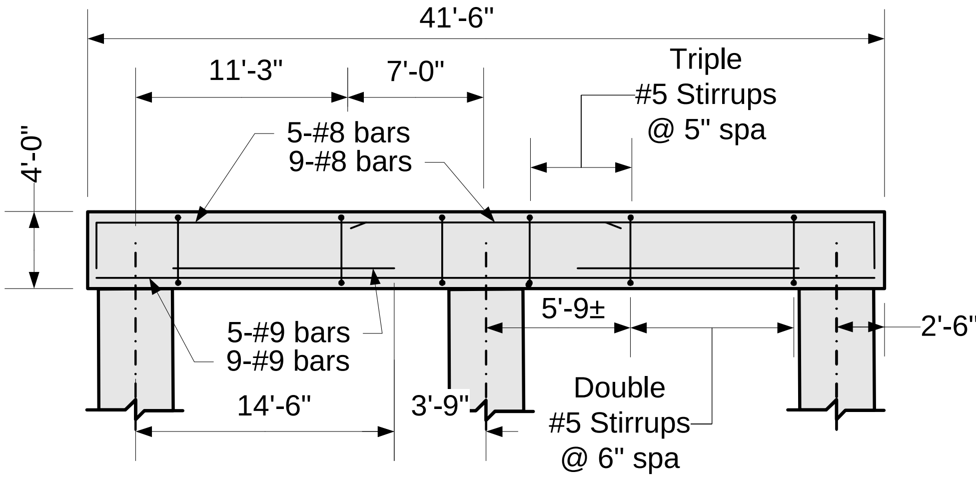
$$col_{face} := \frac{col_w}{2} \cdot \frac{1}{col_{spa}}$$

$col_{face} = 0.11$ % along cap

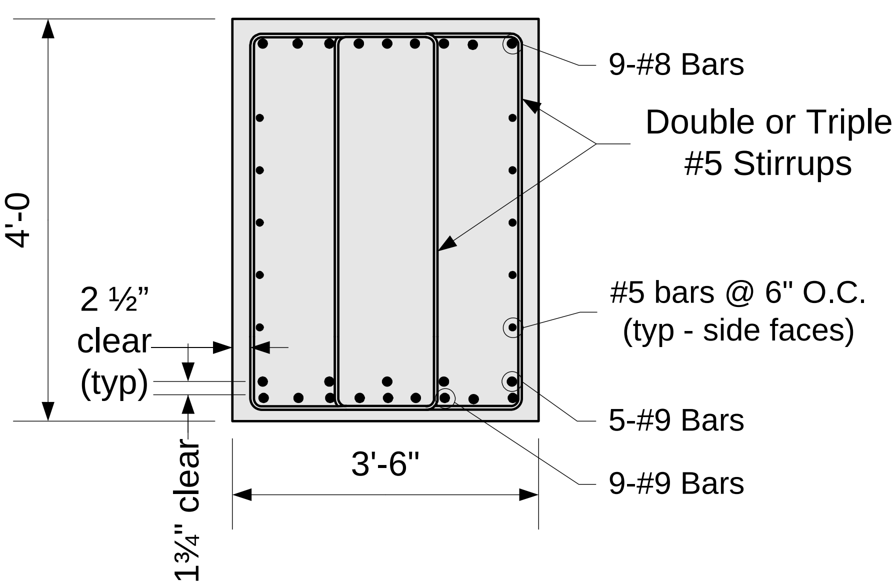
$Mu_{negative}(col_{face}) = -378.37$ kip-ft

$Ms_{negative}(col_{face}) = -229.74$ kip-ft

E13-2.7 Reinforcement Summary



<u>Figure E13-2.7-1</u>
Cap Reinforcement - Elevation View



<u>Figure E13-2.7-2</u>
Cap Reinforcement - Section View

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Table of Contents

14.1 Introduction

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

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

WisDOT policy item:

Retaining walls (such as MSE walls with precast concrete panel facing) that are susceptible to damage from vehicular impact shall be protected by a roadway barrier.

14.1.1 Wall Development Process

Overall, the wall development process requires an iterative collaboration between WisDOT Regions, Structures Design Section, Geotechnical 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 unless otherwise specified below. For a continuous wall consisting of various wall types, the numbering system should include unit numbers so that the numbering appears as R-XX-XXX-001, R-XX-XXX-002, and so on. The first two digits represent the county the wall is located in and the next set(s) of digits represent the undivided wall.

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

- <u>Proprietary retaining walls</u> (e.g., modular block gravity walls, MSE walls, etc.):
  - MSE walls having a maximum height of less than 5.5 ft. measured from the bottom of wall or top of leveling pad to top of wall are deemed to be "minor retaining walls" and do not require an R number. Refer to *FDM 11-55-5.2* for more information.
  - Modular block gravity walls having a maximum height of less than 4.0 ft. measured from the bottom of wall or top of leveling pad to top of wall are deemed to be "minor retaining walls" and do not require an R number. Refer to *FDM 11-55-5.2* for more information.

- Non-proprietary walls (e.g., cast-in-place, sheet pile, and all other wall types other than those previously referenced):

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

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

14.2 Wall Types

Retaining walls can be divided into many categories as discussed below.

Conventional Walls

Retaining walls can be divided into gravity, semi-gravity, and non-gravity cantilever or anchored walls. A brief description of these walls is presented in 14.2.1 and 14.2.2 respectively.

Miscellaneous types of walls including multi-tiered walls, and hybrid or composite walls are also used by combining the wall types mentioned in the previous paragraph. These walls are used only under special project requirements. These walls are briefly discussed in 14.2.3, but the design requirements of these walls will not be presented in this chapter. In addition, some walls are also used for temporary shoring and discussed briefly in 14.2.4.

Permanent or Temporary Walls

All walls can be divided into permanent or temporary walls, depending on project application. Permanent walls have a typical designed life of 75 years. The temporary walls are designed for a service life of 3 years, or the intended project duration, whichever is greater. Temporary wall systems have less restrictive requirements for construction, material and aesthetics.

Fill Walls or Cut Walls

A retaining wall can also be classified as a fill wall, or a cut wall. This description is based on the nature of the earthwork required to construct the wall. If the roadway cross-sections (which include the wall) indicate that existing earth/soil must be removed (excavated) to install the wall, it is considered a ‘cut’ wall. If the roadway cross-sections indicate that earth fill will be placed behind the wall, with little excavation, the wall is considered a ‘fill’ wall. Sometimes wall construction requires nearly equal combinations of earth excavation and earth fill, leading to the nomenclature of a ‘cut/fill’ wall.

Bottom-up or Top-down Constructed Walls

This wall classification method refers to the method in which a wall is constructed. If a wall is constructed from the bottom of the wall, upward to the top, it is considered a bottom-up type of wall. Examples of this include CIP cantilever, MSE and modular block walls. Bottom-up walls are generally the most cost effective type. If a wall is constructed downward, from the top of the wall to the bottom, it is considered a top-down type of wall. This generally requires the insertion of some type of wall support member below the existing ground, and then excavation in front of the wall to the bottom of the exposed face. Examples of this include soil nail, cantilever sheet pile and anchored sheet pile walls. These walls are generally used when excavation room is limited.

Proprietary or Non-Proprietary

Some retaining walls have prefabricated modules or components that are proprietary in nature. Based on the use of proprietary components, walls can be divided into the categories of proprietary and non-proprietary wall systems as defined in 14.1.1.

A proprietary retaining wall system is considered as a patented or trademarked retaining wall system or a wall system comprised of elements/components that are protected by a trade name, brand name, or patent and are designed and supported by the manufacturer. MSE walls, modular block gravity walls, bin, and crib walls are considered proprietary walls because these walls have components which are either patented or have trademarks.

Proprietary walls require preapproval and appropriate special provisions. The preapproval requirements are discussed in 14.16 of this chapter. Proprietary walls also have special design requirements for the structural components, and are discussed in further detail within each specific wall design section. Most MSE, modular block, bin or crib walls require pre-approval and/or special provisions.

A non-proprietary retaining wall is fully designed and detailed by the designer or may be design-build. A non-proprietary retaining wall system may contain proprietary elements or components as well as non-proprietary elements and components. CIP cantilever walls, rock walls, soil nail walls and non-gravity walls fall under this category.

Wall classification is shown in Table 14.2-1 and is based on wall type, project function category, and method of construction.

14.2.1 Gravity Walls

Gravity walls are considered externally stabilized walls as these walls use self weight to resist lateral pressures due to earth and water. Gravity walls are generally subdivided into mass gravity, semi-gravity, modular gravity, mechanically stabilized reinforced earth (MSE), and in-situ reinforced earth wall (soil nailing) categories. A schematic diagram of the various types of gravity walls is included in Figure 14.2-1.

14.2.1.1 Mass Gravity Walls

A mass gravity wall is an externally stabilized, cast-in-place rigid gravity wall, generally trapezoidal in shape. The construction of these walls requires a large quantity of materials so these are rarely used except for low height walls less than 8.0 feet. These walls mainly rely on self weight to resist external pressures and their construction is staged as bottom up construction, mostly in fill or cut/fill situation.

14.2.1.2 Semi-Gravity Walls

Semi-gravity walls resist external forces by the combined action of self weight, weight of soil above footing and the flexural resistance of the wall components. A cast-in-place (CIP) concrete cantilever wall is an example and consists of a reinforced concrete stem and a base footing. These walls are non-proprietary.

Cantilever walls are best suited for use in areas exhibiting good bearing material. When bearing or settlement is a problem, these walls can be founded on piles or foundation improvement may be necessary. Walls exceeding 28 feet in height are provided with counter-forts or buttress slabs. Construction of these walls is staged as bottom-up construction and mostly constructed in fill situations. Cantilever walls are more suited where MSE walls are not feasible, although these walls are generally costlier than MSE walls.

14.2.1.3 Modular Gravity Walls

Modular walls are also known as externally stabilized gravity walls as these walls resist external forces by utilizing self weight. Modular walls have prefabricated modules/components which are considered proprietary. The construction is bottom-up construction mostly used in fill situations.

14.2.1.3.1 Modular Block Gravity Walls

Modular block concrete facings are used without soil reinforcement to function as an externally stabilized gravity wall. The modular blocks are prefabricated dry cast or wet cast concrete blocks and the blocks are stacked vertically or slightly battered to resist external forces. The concrete blocks are either solid concrete or hollow core concrete blocks. The hollow core concrete blocks are filled with crushed aggregates or sand. Modular block gravity walls are limited to a maximum design height of 8 feet under optimum site geometry and soils conditions, but site conditions generally dictate the need for MSE walls when design heights are greater than 5.5 feet. Walls with a maximum height of less than 4 feet are deemed as “minor retaining walls” and do not require an R number. Refer to *FDM 11-55-5.2* for more information. The modular blocks are proprietary and vary in sizes.

14.2.1.3.2 Prefabricated Bin, Crib and Gabion Walls

<u>Bin Walls</u>: Concrete and metal bin walls are built of adjoining open or closed faced bins and then filled with soil/rocks. Each metal bin is comprised of individual members bolted. The concrete bin wall is comprised of prefabricated interlocking concrete modules. These wall systems are proprietary wall systems.

<u>Crib Walls</u>: Crib walls are constructed of interlocking prefabricated units of reinforced or unreinforced concrete or timber elements. Each crib is comprised of longitudinal and transverse members. Each unit is filled with free draining material. These wall systems are proprietary wall systems.

<u>Gabion Walls</u>**:** Gabion walls are constructed of steel wire baskets filled with selected rock fragments and tied together. Gabions walls are flexible, free draining and easy to construct. These wall systems are proprietary wall systems. Maximum heights are normally less than 21 feet. These walls are desirable where equipment access is limited. The wires used for constructing gabions baskets must be designed with adequate corrosion protection.

14.2.1.4 Rock Walls

Rock walls are also known as 'Rockery Walls'. These types of gravity walls are built by stacking locally available large stones or boulders into a trapezoid shape. These walls are highly flexible and height of these walls is generally limited to approximately 8.0 feet. A layer of gravel and geotextile is commonly used between the stones and the retained soil. These walls can be designed using the *FHWA Rockery Design and Construction Guideline*.

14.2.1.5 Mechanically Stabilized Earth (MSE) Walls:

Mechanically Stabilized Earth (MSE) walls include a selected soil mass reinforced with metallic or geo-synthetic reinforcement. The soil reinforcement is connected to a facing element to prevent the reinforced soil from sloughing. Construction of these walls is staged as bottom-up construction. These can be constructed in cut and fill situations, but are better suited to fill sites. MSE walls are normally used for wall heights between 10 to 40 feet. A brief description of various types of MSE walls is given below:

<u>Precast Concrete Panel MSE Walls</u>: These types of walls employ a metallic strip or wire grid reinforcement connected to precast concrete panels to reinforce a selected soil mass. The concrete panels are usually 5'x5' or 5'x10' size panels. These walls are proprietary wall systems.

<u>Modular Block Facing MSE Wall</u>: Prefabricated modular concrete block walls consist of almost vertically stacked concrete modular blocks and the soil reinforcement is secured between the blocks at predetermined levels. Metallic strips or geogrids are generally used as soil reinforcement to reinforce the selected soil mass. Concrete blocks are either solid or hollow core blocks. The hollow core blocks are filled with aggregates or sand. These types of walls are proprietary wall systems.

<u>Geotextile/Geogrids/Welded Wire Faced MSE Walls</u>: These types of MSE walls consist of compacted soil layers reinforced with continuous or semi-continuous geotextile, geogrid or welded wire around the overlying reinforcement. The wall facing is formed by wrapping each layer of reinforcement around the overlying layer of backfill and re-embedding the free end into the backfill. These types of walls are used for temporary or permanent applications. Permanent facings include shotcrete, gunite, galvanized welded wire mesh, cast-in-place concrete or prefabricated concrete panels.

14.2.1.6 Soil Nail Walls

Soil nail walls are internally stabilized cut walls that use in-situ reinforcement for resisting earth pressures. The large diameter rebars (generally #10 or greater) are typically used for the reinforcement. The construction of soil nail walls is staged top-down and soil nails are installed after each stage of excavation. Shotcrete can be applied as a facing. The facing of a soil nail wall is typically covered with vertical drainage strips located over the nail then covered with shotcrete. Soil nailing walls are used for temporary or permanent construction. Specialty contractors are required when constructing these walls. Soil nail walls have been installed to heights of 60.0 feet or more but there have only been a few soil nail walls constructed on WisDOT projects.

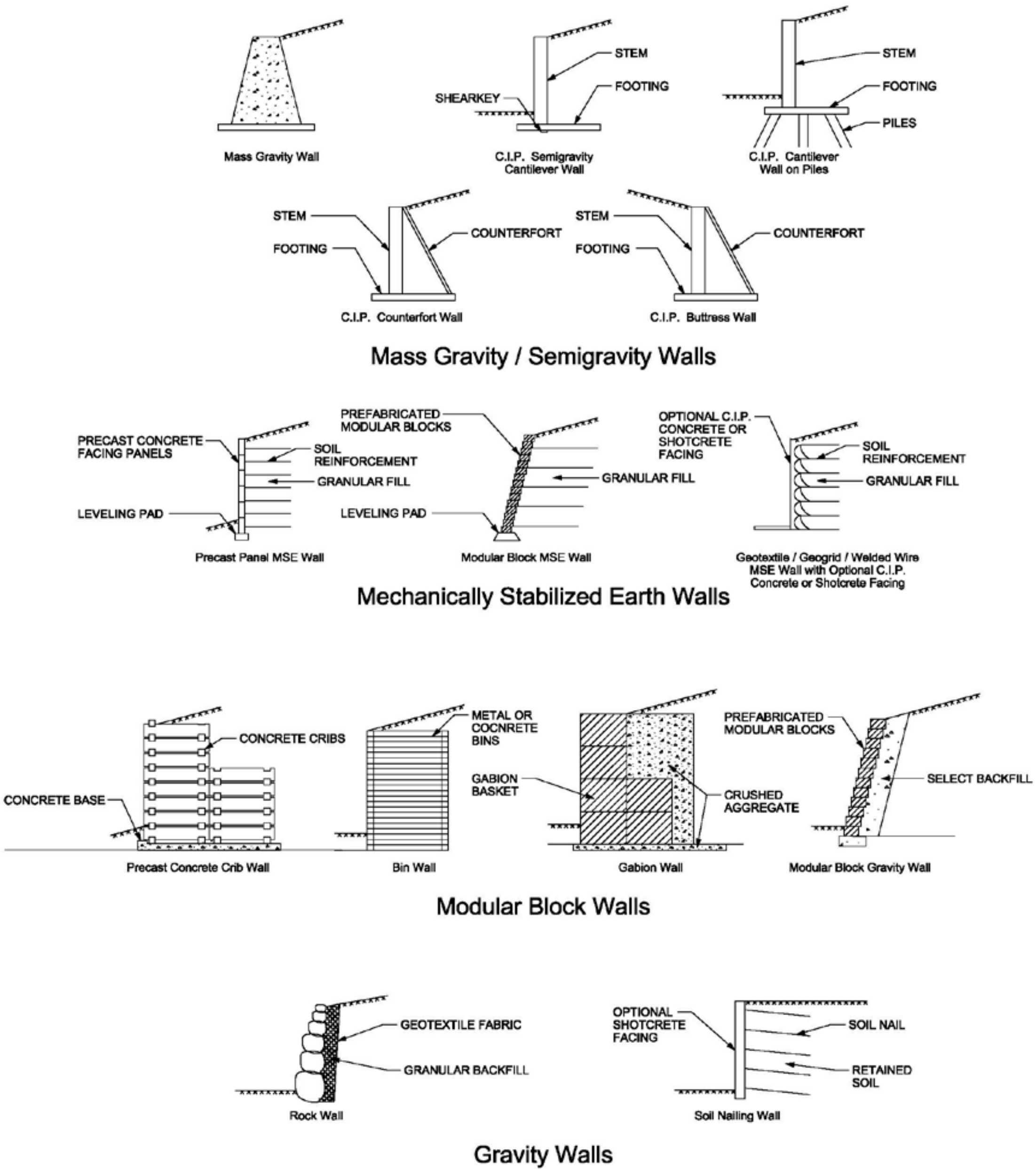


Figure 14.2-1
Gravity Walls

14.2.2 Non-Gravity Walls

Non-gravity walls are classified into cantilever and anchored wall categories. These walls are considered as externally stabilized walls and used in cut situations. The walls include sheet pile, post and panel, tangent and secant pile type with or without anchors. Figure 14.2-2 shows common types of non-gravity walls.

14.2.2.1 Cantilever Walls

These types of walls derive lateral resistance through embedment of vertical elements into natural ground and the flexure resistance of the structural members. They are used where excavation support is needed in shallow cut situations.

Cantilever Sheet Pile Walls: Cantilever sheet pile walls consist of interlocking steel panels, driven into the ground to form a continuous sheet pile wall. The sheet piles resist the lateral earth pressure utilizing the passive resistance in front of the wall and the flexural resistance of the sheet pile. Most sheet pile walls are less than 15 feet in height.

Soldier Pile Walls: These types of walls are non gravity wall systems that derive lateral resistance and moment capacity through embedment of vertical members (soldier piles) into natural ground in cut situations. The vertical elements may be drilled or driven steel or concrete members. The soil behind the wall is retained by lagging. The lagging may be steel, wood, or concrete.

Post and Panel Walls: These types of walls are comprised of vertical elements (usually H piles) and concrete panels which extend between vertical elements. The panels are usually constructed of precast reinforced concrete or precast prestressed concrete. These walls should be considered when disturbance to the site is critical. These are also suitable for site where rock is encountered near surface. Post and panel walls are constructed from bottom up.

Tangent and Secant Pile Walls: A tangent pile wall consists of a single row of reinforced concrete piles (drilled) installed in the ground. Each pile touches the adjacent pile tangentially. The concrete piles are reinforced using a single steel beam or a cage of reinforcing bars. A secant wall, generally, consists of a single row of overlapping and alternating reinforced and unreinforced piles drilled into the ground. Secant and tangent wall systems are used to hold earth and water where water tightness is important, and lowering of the water table is not desirable.

14.2.2.2 Anchored Walls

Anchored walls are externally stabilized non-gravity cut walls. Anchored walls are essentially the same as cantilever walls except that these walls utilize anchors (tiebacks) to extend the wall heights beyond the design limit of the cantilever walls. These walls require less toe embedment than cantilever walls.

These walls derive lateral resistance by embedment of vertical wall elements into firm ground and by anchorages. Most commonly used anchored walls are anchored sheet pile walls and the soldier pile walls. Tangent and secant walls can also be anchored with tie backs and used as anchored walls. The anchors can be attached to the walls by tie rods, bars or wired tendons. The anchoring device is generally a deadman, screw-type, or grouted tieback anchor. Anchored walls can be built to significant heights.

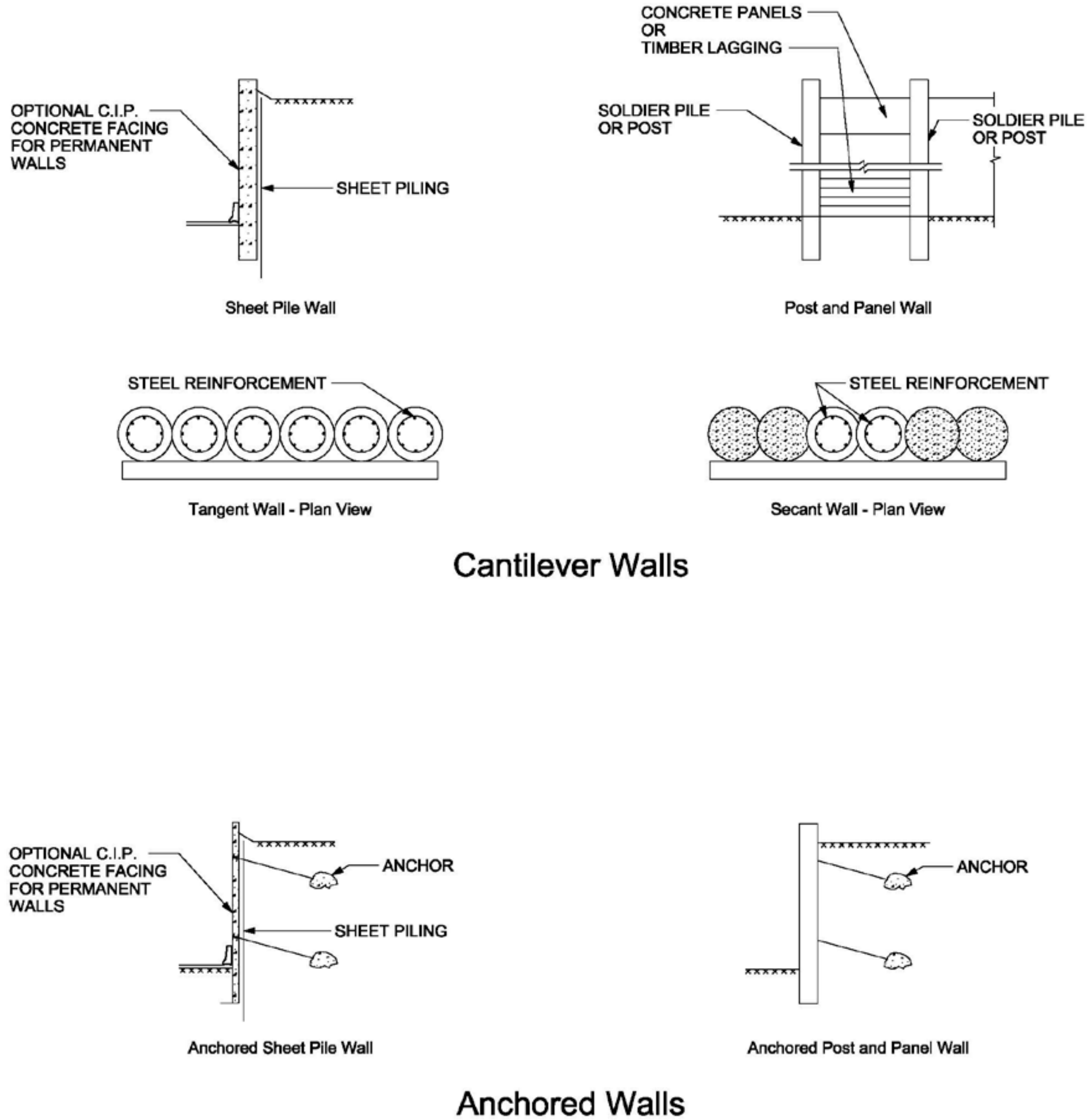


Figure 14.2-2
Non-Gravity Walls

14.2.3 Tiered and Hybrid Wall Systems

A tiered wall system is a series of two or more walls, each higher wall set back from the underlying walls. The upper wall exerts an additional surcharge on the lower lying wall and requires special design attention. The design of these walls has not been discussed in this chapter. Hybrids wall systems combine wall components from two or more different wall systems and provide an alternative to a single type of wall used in cut or fill locations. These types of walls require special design attention as components of these walls require different magnitudes of deformation to develop loading resistance. The design of such walls will be on a case-by-case basis, and is not discussed in this chapter.

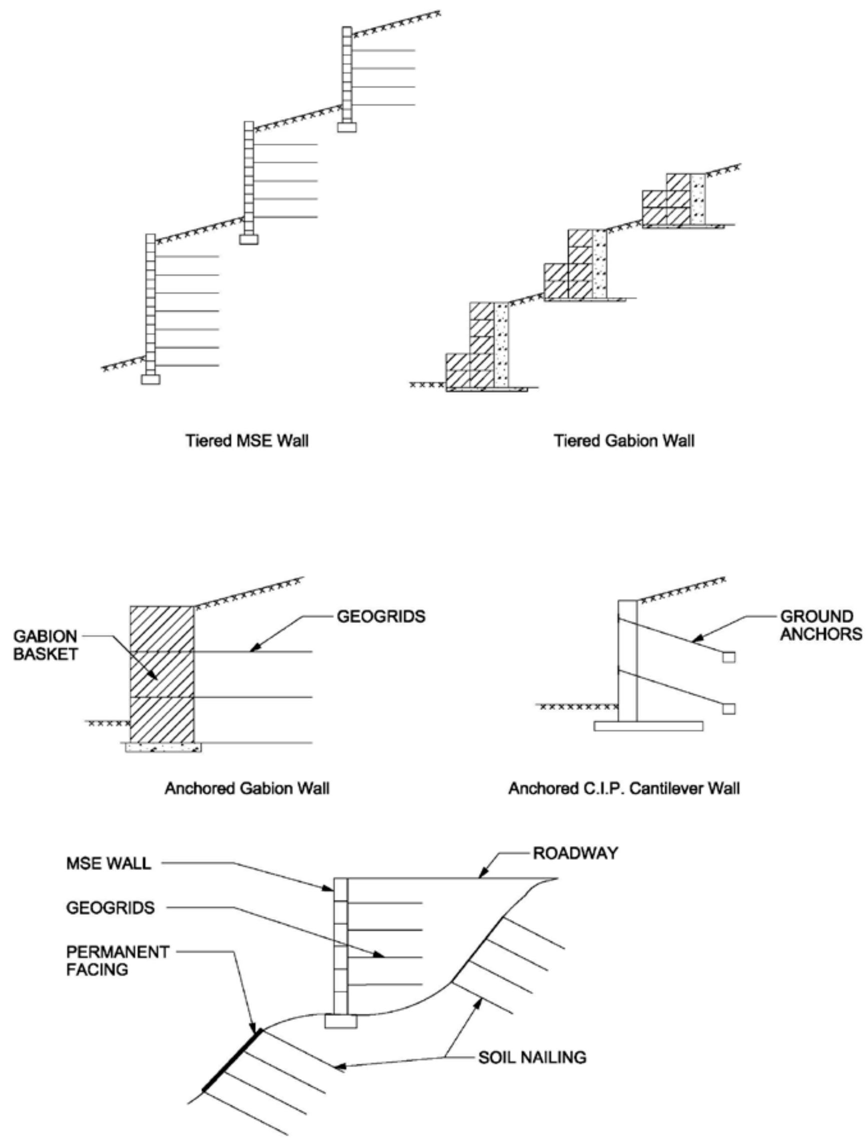
Some examples of tiered and hybrid walls systems are shown in Figure 14.2-3.

14.2.4 Temporary Shoring

Temporary shoring is used to protect existing transportation facilities, utilities, buildings, or other critical features when safe slopes cannot be made for structural excavations. Shoring may be required within the limits of structures or on the approach roadway due to grade changes or staged construction. Shoring should not be required nor paid for when used primarily for the convenience of the contractor. Temporary shoring is designed by the contractor. MSE walls with flexible facings and sheet pile walls are commonly used for temporary shoring.

14.2.5 Wall Classification Chart

A wall classification chart has been developed and shown as Table 14.2-1.



<u>Figure 14.2-3</u>
Tiered & Hybrid Wall Systems

Wall Category	Wall Sub-Category	Wall Type	Typical Construction Concept	Proprietary
Gravity	Mass Gravity	CIP Gravity	Bottom Up (Fill)	No
	Semi-Gravity	CIP Cantilever	Bottom Up (Fill)	No
	Reinforced Earth	MSE Walls- Precast Panel, Modular Blocks, Geogrid/ Geo-textile/Wire-Faced	Bottom Up (Fill)	Yes
	Modular Gravity	Modular Blocks, Gabion, Bin, Crib	Bottom Up/(Fill)	Yes
	In-situ Reinforced	Soil Nailing	Top Down (Cut)	No
Non-Gravity	Cantilever	Sheet Pile, Soldier Pile, Tangent/Secant	Top Down (Cut)	No
	Cantilever	Post and Panel, Tangent/Secant	Bottom up(Fill)	No
	Anchored	Anchored Sheet Pile, Soldier Pile, Tangent/Secant	Top Down (Cut)	No

Table 14.2-1

Wall Classification

14.3 Wall Selection Criteria

14.3.1 General

The objective of selecting a wall system is to determine an appropriate wall system that is practical to construct, structurally sound, economic, aesthetically pleasing, environmentally consistent with the surroundings, and has minimal maintenance problems.

With the development of many new wall systems, designers have the choice of selecting many feasible wall systems that can be constructed on a given highway project. Designers are encouraged to evaluate several feasible wall systems for a particular project where wall systems can be economically constructed. After consideration of various wall types, a single type should be selected for final analyses and design. Wall designers must consider the general design concepts described in section 14.4 and specific wall design requirements described in 14.5 thru 14.11 of this chapter, and key wall selection factors discussed in this section.

In general, selection of a wall system should include, but not limited to the key factors described in this section for consideration when generating a list of acceptable retaining wall systems for a given site.

14.3.1.1 Project Category

The designer should consider if the wall system is permanent or temporary.

14.3.1.2 Cut vs. Fill Application

Due to construction techniques and base width requirements for stability, some wall types are better suited for cut sections where as others are suited for fill or fill/cut situations. The key considerations are the amount of excavation or shoring, overall wall height, proximity of wall to other structures, and right-of-way width available. The site geometry should be evaluated to define site constraints. These constraints will generally dictate if fill, fill/cut or cut walls are required.

Cut Walls

Cut walls are generally constructed from the top down and used for both temporary and permanent applications. Cantilever sheet pile walls are suitable for shallower cuts. If a deeper cut is required to be retained, a key question is to determine the availability of right-of-way (ROW). Subsurface conditions such as shallow bedrock also enter into considerations of cut walls. Anchored walls, soil nail walls, and anchored soldier pile walls may be suitable for deeper cuts although these walls require either a larger permanent easement or permanent ROW.

Fill walls

Walls constructed in fill locations are typically used for permanent construction and may require large ROW to meet the base width requirements. The necessary fill material may be required to be granular in nature. These walls use bottom up construction and have typical cost effective ranges. Surface conditions must also be considered. For instance, if soft compressible soils are present, walls that can tolerate larger settlements and movements must be considered. MSE walls are generally more economical for fill locations than CIP cantilever walls.

Cut/fill Walls

CIP cantilever and prefabricated modular walls are most suitable in cut/fill situations as the walls are built from bottom up, have narrower base widths and these walls do not rely on soil reinforcement techniques to provide stability. These types of walls are suitable for both cut or fill situations.

14.3.1.3 Site Characteristics

Site characterization should be performed, as appropriate, to provide the necessary information for the design and construction of retaining wall systems. The objective of this characterization is to determine composition and subsurface soil/rock conditions, define engineering properties of foundation material and retained soils, establish groundwater conditions, determine the corrosion potential of the water, identify any discontinuities or geotechnical issues such as poor bearing capacity, large settlement potential, and/or any other design and construction problems.

Site characterization mainly includes subsurface investigations and analyses. WisDOT's Geotechnical Unit generally completes the investigation and analyses for all in-house wall design work.

14.3.1.4 Miscellaneous Design Considerations

Other key factors that may influence wall selection include height limitations for specific systems, limit of wall radius on horizontal alignment, and whether the wall is a component of an abutment.

Foundation conditions that may govern the wall selection are bearing capacity, allowable lateral and vertical movements, tolerable settlement and differential movement of retaining wall systems being designed, susceptibility to scour or undermining due to seepage, and long-term maintenance.

14.3.1.5 Right of Way Considerations

Availability of ROW at a site may influence the selection of wall type. When a very narrow ROW is available, a sheet pile wall may be suitable to support an excavation. In other cases, when walls with tiebacks or soil reinforcement are considered, a relatively large ROW may be required to meet wall requirements. Availability of vertical operating space may influence wall selection where piling installation is required and there is not enough room to operate driving equipment.

Section 11-55-5 of the FDM describes the ROW requirement for retaining walls. It requires that all segments of a retaining wall should be under the control of WisDOT. No improvements or utility construction should be allowed in the ROW area of the retaining wall systems.

14.3.1.6 Utilities and Other Conflicts

Feasibility of some wall systems may be influenced by the presence of utilities and buried structures. MSE, soil nailing and anchored walls commonly have conflict with the presence of utilities or buried underground structures. MSE walls should not be used where utilities must stay in the reinforcement zone.

14.3.1.7 Aesthetics

In addition to being functional and economical, the walls should be aesthetically pleasing. Wall aesthetics may influence selection of a particular wall system. However, the aesthetic treatment should complement the retaining wall and not disrupt the functionality or selection of wall type. All permanent walls should be designed with due considerations to the wall aesthetics. Each wall site must be investigated individually for aesthetic needs. Temporary walls should generally be designed with little consideration to aesthetics. Chapter 4 - Aesthetics presents structures aesthetic requirements.

14.3.1.8 Constructability Considerations

Availability of construction material, site accessibility, equipment availability, form work and temporary shoring, dewatering requirements, labor considerations, complicated alignment changes, scheduling consideration, speed of construction, construction staging/phasing and maintaining traffic during construction are some of the important key factors when evaluating the constructability of each wall system for a specific site project.

In addition, it should also be ensured that the temporary excavation slopes used for wall construction are stable as per site conditions and meet all safety requirements laid by Occupation and Safety Health Administration (OSHA).

14.3.1.9 Environmental Considerations

Selection of a retaining wall system is influenced by its potential environmental impact during and after construction. Some of the environmental concerns during construction may include excavation and disposal of contaminated material at the project site, large quantity of water, corrosive nature of water, vibration impacts, noise abatement and pile driving constraints.

14.3.1.10 Cost

Cost of a retaining wall system is influenced by many factors that must be considered while estimating preliminary costs. The components that influence cost include excavation, structure, procurement of additional easement or ROW, drainage, disposal of unsuitable material, traffic maintenance etc. Maintenance cost also affects overall cost of a retaining wall system. The retaining walls that have least structural cost may not be the most economical walls. Wall selection should be based on overall cost. When feasible, MSE Walls and modular block gravity walls cost less than other walls

14.3.1.11 Mandates by Other Agencies

In certain project locations, other agency mandates may limit the types of wall systems considered.

14.3.1.12 Requests made by the Public

A Public Interest Finding could dictate the wall system to be used on a specific project.

14.3.1.13 Railing

For safety reasons most walls will require a protective railing. The railing will usually be located behind the wall. The roadway designer will generally determine whether a pedestrian or non-pedestrian railing is required and what aesthetic considerations are needed.

14.3.1.14 Traffic barrier

A traffic barrier should be installed if vehicles, bicycles, or pedestrians are likely to be present on top of the wall. The roadway designer generally determines the need for a traffic barrier.

14.3.2 Wall Selection Guide Charts

Table 14.3-1 and Table 14.3-2 summarize the characteristics for the various wall types that are normally considered during the wall selection process. The tables also present some of the advantages, disadvantages, cost effective height range and other key selection factors. A wall designer can use these tables and the general wall selection criteria discussed in 14.3.1 as a guide. Designers are encouraged to contact the Structures Design Section if they have any questions relating to wall selection for their project.

Wall Type	Temp	Perm	Cost Effective Height (ft)	Req'd. ROW	Advantages	Disadvantages
Concrete Gravity		√	3-10	.5H-.7H	• Durable • Meets aesthetic requirement • Requires small quantity of select backfill	• High cost • May need deep foundation • Longer const. time
Reinforced CIP Cantilever		√	6-28	.4H-.7H	• Durable meets aesthetic requirement • Requires small quantity of select backfill	• High cost • May need deep foundation • Longer const. time & deeper embedment
Reinforced CIP Counterfort		√	26 -40	0.4H-0.7H	• Durable • Meets aesthetic requirement • Requires small back fill quantity	• High cost • May need deep foundation • Longer const. time & deeper embedment
Concrete Modular Block		√	3-8	.4H-.7H	• Does not require skilled labor or specialized equipment	• Height limitations
Metal Bin		√	6 -20	.4H-.7H	• Does not require skilled labor or special equipment	• Difficult to make height adjustment in the field
Concrete Crib		√	6-20	.4H-.7H	• Does not require skilled labor or specialized equipment	• Difficult to make height adjustment in the field
Gabion		√	6-20	.4H-.7H	• Does not require skilled labor or specialized equipment	• Need large stone quantities • Significant labor
MSE Wall (precast concrete panel with steel reinforcement)		√	10-35	.7H-1.0H	• Does not require skilled labor or specialized equipment	• Requires use of select backfill
MSE Wall (modular block and geo-synthetic reinforcement)		√	6-22	.7H-1.0H	• Does not require skilled labor or specialized equipment	• Requires use of select backfill
MSE Wall (geotextile/ geogrid / welded wire facing)	√	√	6-35	.7H-1.0H	• Does not require skilled labor or specialized equipment	• Requires use of select backfill

Table 14.3-1
Wall Selection Chart for Gravity Walls

Wall Type	Temp	Perm	Cost Effective Height (ft)	Req'd. ROW	Water Tightness	Advantages	Disadvantages
Sheet Pile	√	√	6-15	minimal	fair	• Rapid construction • Readily available	• Deep foundation may be needed • Longer construction time
Post & Panel	√	√	6-28	.2H-.5H	poor	• Easy construction • Readily available	• High cost • Deep foundation may be needed • Longer construction time
Tangent Pile		√	20 -60	.4H-.7H	good	• Adaptable to irregular layout • Can control wall stiffness	• High cost • Deep foundation may be needed • Longer construction
Secant Pile Wall		√	14-60	.4H-.7H	fair	• Adaptable to irregular layout • Can control wall stiffness	• Difficult to make height adjustment in the field • High cost
Anchored Wall	√	√	15-35	.4H-.7H	fair	• Rapid construction	• Difficult to make height adjustment in the field
Soil Nail Wall	√	√	6-20	.4H-.7H	fair	• Option for top-down	• Cannot be used in all soil types • Cannot be used below water table • Significant labor

Table 14.3-2
Wall Selection Chart for Non-Gravity Walls

14.4 General Design Concepts

This section covers the general design standards and criteria to be used for the design of temporary and permanent gravity and non-gravity walls including proprietary and non-proprietary wall systems.

The design criteria for tiered walls that retain other walls or hybrid walls systems requiring special design are not covered specifically in this section.

14.4.1 General Design Steps

The design of wall systems should follow a systematic process applicable for all wall systems and summarized below:

1. Basic Project Requirement: This includes determination of wall alignment, wall geometry, wall function, aesthetic, and project constraints (e.g. right of way, easement during construction, environment, utilities etc) as part of the wall development process described in 14.1.

2. Geotechnical Investigation: Subsurface investigation and analyses should be performed in accordance with 14.4.4 and Chapter 10 - Geotechnical Investigation to develop foundation and fill material design strength parameters and foundation bearing capacity.

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

4. Wall Loading: Determine all applicable loads likely to act on the wall as discussed in 14.4.5.3.

5. Initial Wall Sizing: This step requires initial sizing of various wall components and establishing wall batter which is wall specific and described under each specific wall designs discussed in 14.5 thru 14.13.

6. Wall Design Requirements: Design wall systems using design standards and service life criteria and the *AASHTO Load and Resistance Factor Design* (*AASHTO LRFD*) requirements discussed in 14.4.1 and 14.4.2.

7. Perform external stability, overall stability, and wall movement checks discussed in 14.4.7. These checks will be wall specific and generally performed by the Geotechnical Engineer of record. The stability checks should be performed using the performance limits, load combinations, and the load/resistance factors per *AASHTO LRFD* requirements described in 14.4.5.5 and 14.4.5.6 respectively.

8. Perform internal stability and structural design of the individual wall components and miscellaneous components. These computations are performed by the Designer. For proprietary walls, internal stability is the responsibility of the contractor/supplier

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

14.4.2 Design Standards

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

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

14.4.3 Design Life

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

14.4.4 Subsurface Exploration

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

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

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

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

14.4.5 Load and Resistance Factor Design Requirements

14.4.5.1 General

In the LRFD process, wall stability is checked as part of the design process for anticipated failure modes for various types of walls at specified limit states, and the wall components are sized accordingly.

To evaluate the limit states, all applicable design loads are computed as nominal or un-factored loads, than factored using a load factor and grouped to consider the force effect of all loads and load combinations in accordance with **LRFD [3.4.1]**. The factored loads are compared with the factored resistance as part of the stability check in accordance with **LRFD [11.5]** such that the factored resistance is not less than factored loads as presented in **LRFD [1.3.2.1]**

$$Q = \sum \eta_i \gamma_I Q_i \leq \phi R_n = R_r$$ **LRFD [1.3.2.1-1]**

Where:

η_I	=	Load modifier (a function of η_D, η_R, assumed 1.0 for retaining walls)
γ_I	=	Load factor
Q_i	=	Force effect
Q	=	Total factored force effect
ϕ	=	Resistance factor
R_n	=	Nominal resistance
R_r	=	Factored resistance = ϕR_n

14.4.5.2 Limit States

The limit states (as defined in **LRFD [3.4.1]**) that must be evaluated as part of the wall design requirements mainly include (1) Strength limit states; (2) Service limit states; and (3) Extreme Event limit states. The fatigue limit state is not used for retaining walls.

Strength limit state is applied to ensure that walls have adequate strength to resist external stability failure due to sliding, bearing resistance failure, etc. and internal stability failure such as pullout of reinforcement, etc. Evaluation of Strength limit states is accomplished by grouping factored loads and comparing to the reduced or factored soil strengths using resistance factors discussed in 14.4.5.6.

Service limit state is evaluated for overall stability and total or differential settlement checks. Evaluation of the Service limit states is usually performed by using expected service loads assuming a factor of 1.0 for nominal loads, a resistance factor of 1.0 for nominal strengths and elastic analyses.

Extreme Event II limit state is evaluated to design walls for vehicular collision forces. In particular, MSE walls having a traffic barrier at the top are vulnerable to damage due to vehicle collision forces and this case for MSE Walls is discussed further in 14.6.3.10.

14.4.5.3 Design Loads

Retaining walls shall be designed to withstand all applicable loads generally categorized as permanent and transient loads.

Permanent loads include dead load DC due to weight of the structural components and non structural components of the wall, dead load DW loads due to wearing surfaces and utilities, vertical earth pressure EV due to dead load of earth, horizontal earth pressure EH and earth surcharge loads ES. Applied earth pressure and earth pressure surcharge loads are further discussed in 14.4.5.4.

The transient loads include, but are not limited to, water pressure WA, live load surcharge LS, and forces caused by the deformations due to shrinkage SH, creep CR and settlement caused by the foundation SE.

These loads should be computed in accordance with **LRFD [3.4]** and **LRFD [11.0].** Only loads applicable for each specific wall type should be considered in the engineering analyses.

14.4.5.4 Earth Pressure

Determination of earth pressure will depend upon types of wall structure (gravity, semi gravity, reinforced earth wall, cantilever or anchored walls etc), wall movement, wall geometry, wall friction, configuration, retained soil type, ground water conditions, earth surcharge, and traffic and construction related live load surcharge. In general, earth pressure on retaining walls shall be calculated in accordance with **LRFD [3.11.5]**. Earth pressure that will develop on walls includes active, passive or at-rest earth pressure.

Active Earth Pressure

The active earth pressure condition exists when a retaining wall is free to rotate away from the retained backfill. There are two earth pressure theories available for determining the active earth pressure coefficient (k_a); Rankine and Coulomb earth pressure theories. A detailed discussion of Rankine and Coulomb theories can be found in *Foundation Design-Principles and Practices*; by Donald P. Cudoto or *Foundation Analysis and Design*, 5th Edition by Joseph E. Bowles as well as other standard text books on this subject.

Rankine earth pressure makes assumptions that the retained soil has a horizontal surface, the failure surface is a plane and that the wall is smooth (i.e. no friction). Rankine earth pressure theory is the preferred method for developing the active earth pressure coefficient; however, where wall friction is an important consideration or where sloping surcharge loads are considered, Coulomb earth pressure theory may be used. The use of Rankine theory will cause a slight over estimation of K_a, therefore, increasing the pressure on the wall resulting in a more conservative design.

Walls that are cast-in-place (CIP) semi gravity concrete cantilever referred, hereafter, as CIP cantilever, Mechanically Stabilized Earth (MSE), modular block gravity, soil nailing, soldier-pile and sheet-pile walls are typically considered flexible enough to justify using an active earth pressure coefficient.

At-Rest Earth Pressure

In the at-rest earth pressure (K_o) condition, the top of the wall is not allowed to deflect or rotate; therefore, requiring the wall to support the full pressure of the soil behind the wall.

The at-rest earth pressure coefficient shall be used to calculate the lateral earth pressure for non-yielding retaining walls restrained from rotation and/or lateral translation in accordance with **LRFD [3.11.5.2]**. Non-yielding walls include integral abutment walls, or retaining walls resting on bedrock or pile foundation.

Passive Earth Pressure

The development of passive earth pressure (K_p) requires a retaining wall to move into or toward the soil. As with the active earth pressure, Rankine earth pressure is the preferred method to be used to develop passive earth pressure coefficient. The use of Rankine theory will cause an under estimation of K_p, therefore resulting in a more conservative design. Coulomb earth pressure theory may be used if the appropriate conditions exist at a site; however, the designer is required to understand the limitations on the use of Coulomb earth pressure theory as applied to passive earth pressures.

Neglect any contribution from passive earth pressure in stability calculations unless the base of the wall extends below the depth to which foundation soil or rock could be weakened or removed by freeze-thaw, shrink-swell, scour, erosion, construction excavation, or any other means. In wall stability calculations, only the embedment below this depth, known as the effective embedment depth, shall be considered when calculating the passive earth pressure resistance. This is in accordance with **LRFD [11.6.3.5].**

14.4.5.4.1 Earth Load Surcharge

The effect of earth load surcharge including uniform, strip, and point loads shall be computed in accordance with **LRFD [3.11.6.1]** and **LRFD [3.11.6.2].**

14.4.5.4.2 Live Load Surcharge

Increased earth pressure on a wall occurs due to vehicular loading on top of the retained earth including operation of large or heavily-loaded cranes, staged equipment, soil stockpile or material storage, or any surcharge loads behind the walls. Earth pressure from live load surcharge shall be applied when a vehicular load is within one half of the wall height behind the back face of the wall or reinforced soil mass for MSE walls, in accordance with **LRFD [3.11.6.4].** In most cases, surcharge load can be modeled by assuming 2 ft of fill.

WisDOT policy item:

The equivalent height of soils for vehicular loading on retaining walls parallel to the traffic shall be 2.0 feet, regardless of the wall height. For standard unit weight of soil equal to 120 pcf, the resulting live load surcharge is 240 psf. Walls without traffic shall be designed for a live load surcharge of 100 psf to account for construction live loads.

14.4.5.4.3 Compaction Loads

Pressure induced by the compaction load can extend to variable depths due to the total static and dynamic forces exerted by compaction equipments. The effect of increased lateral earth pressure due to compaction loads during construction should be considered when compaction equipment is operated behind the wall. The compaction load surcharge effect is minimized by WISDOT standard specifications that require small walk behind compactors within 3 ft of the wall.

14.4.5.4.4 Wall Slopes

The slopes above and below the wall can significantly affect the earth pressures and wall stability. Slopes above the wall will influence the active earth pressure; slopes at the toe of the wall influences the passive earth pressures. In general, the back slope behind the wall should be no steeper than 2:1 (H:V). Where possible, a 4.0 ft wide horizontal bench should be provided at the front face of the wall.

14.4.5.4.5 Loading and Earth Pressure Diagrams

Loading and earth pressure diagrams are developed to compute nominal (unfactored) loads and moments. All applicable loads described in 14.4.5.3 and 14.4.5 shall be considered for computing nominal loads. For a typical wall, the force diagram for the earth pressure should be developed using a triangular distribution plus additional pressures resulting from earth or live load surcharge, water pressure, compaction etc. as discussed in 14.4.5.4.

The engineering properties for selected fill, concrete and steel are given in 14.4.6. The foundation and retained earth properties are selected as per discussions in 14.4.4 . One of the three cases is generally applicable for the development of loading diagrams and earth pressures:

1. Horizontal backslope with traffic surcharge
2. Sloping backslope
3. Broken backslope

Loading diagrams for CIP cantilever, MSE, modular block gravity, and prefabricated modular walls are shown for illustration. The designer shall develop loading diagrams as applicable.

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.

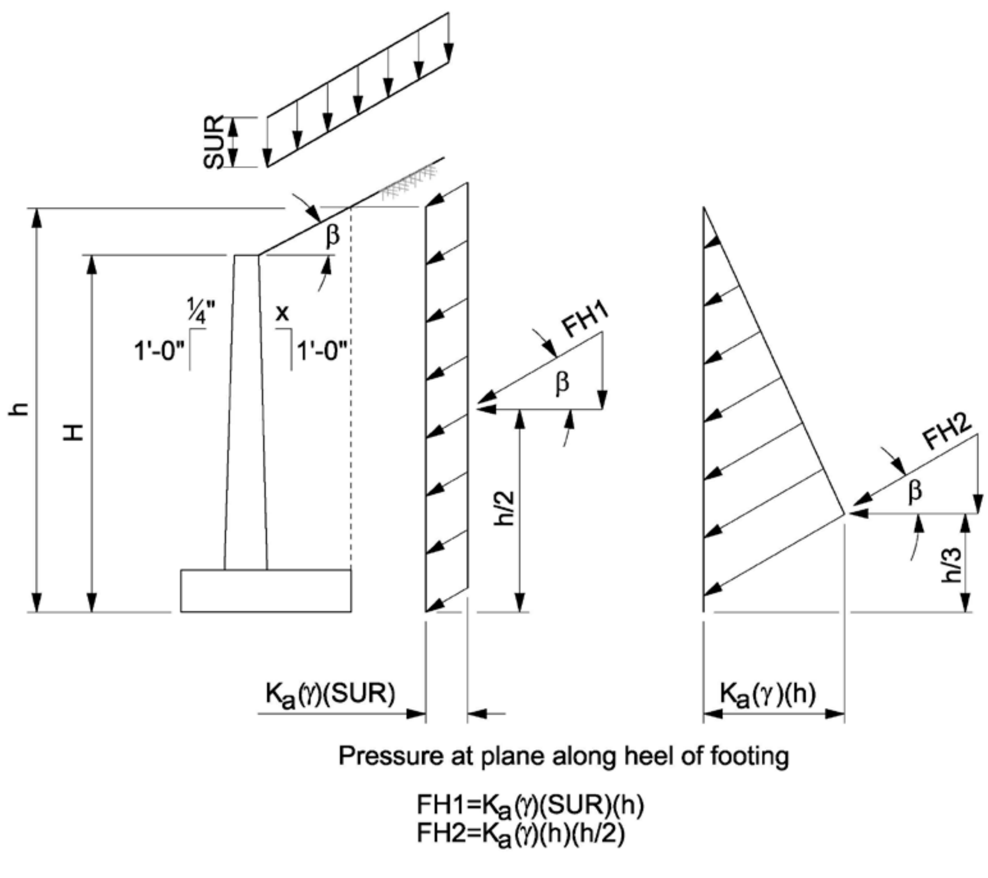


Figure14.4-1

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

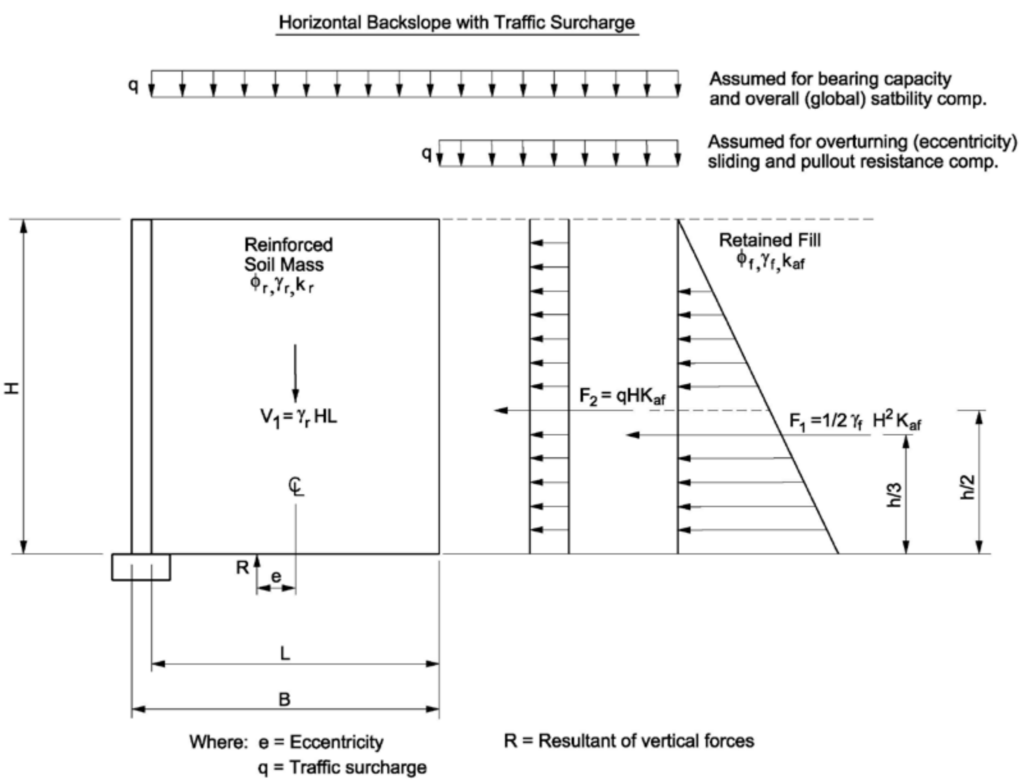


Figure 14.4-2
MSE Walls Earth Pressure for Horizontal Backslope with Traffic Surcharge
(Source *AASHTO LRFD*)

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

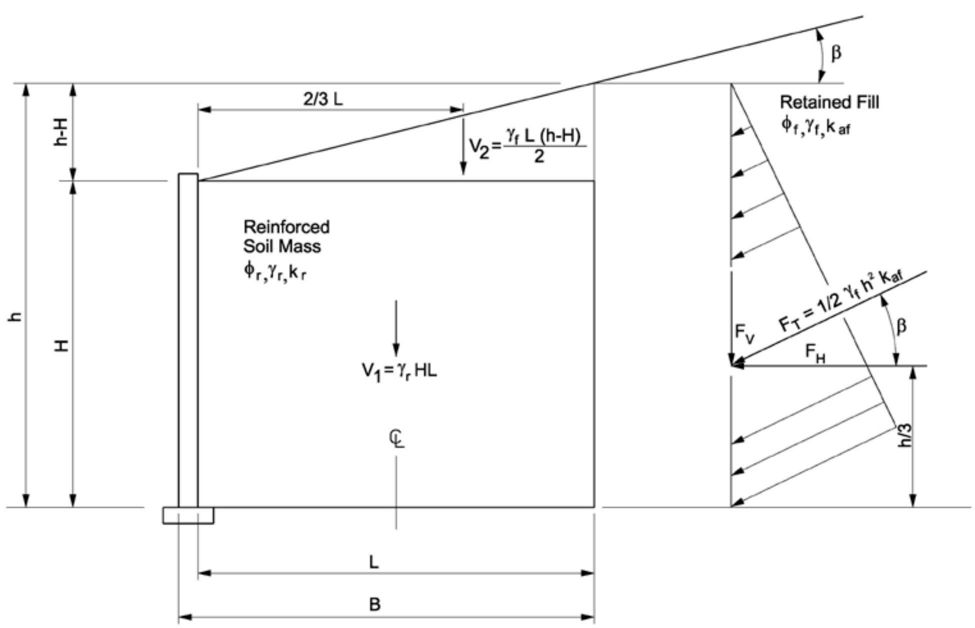


Figure 14.4-3
MSE Walls Earth Pressure for Sloping Backfill
(Source *AASHTO LRFD*)

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:

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

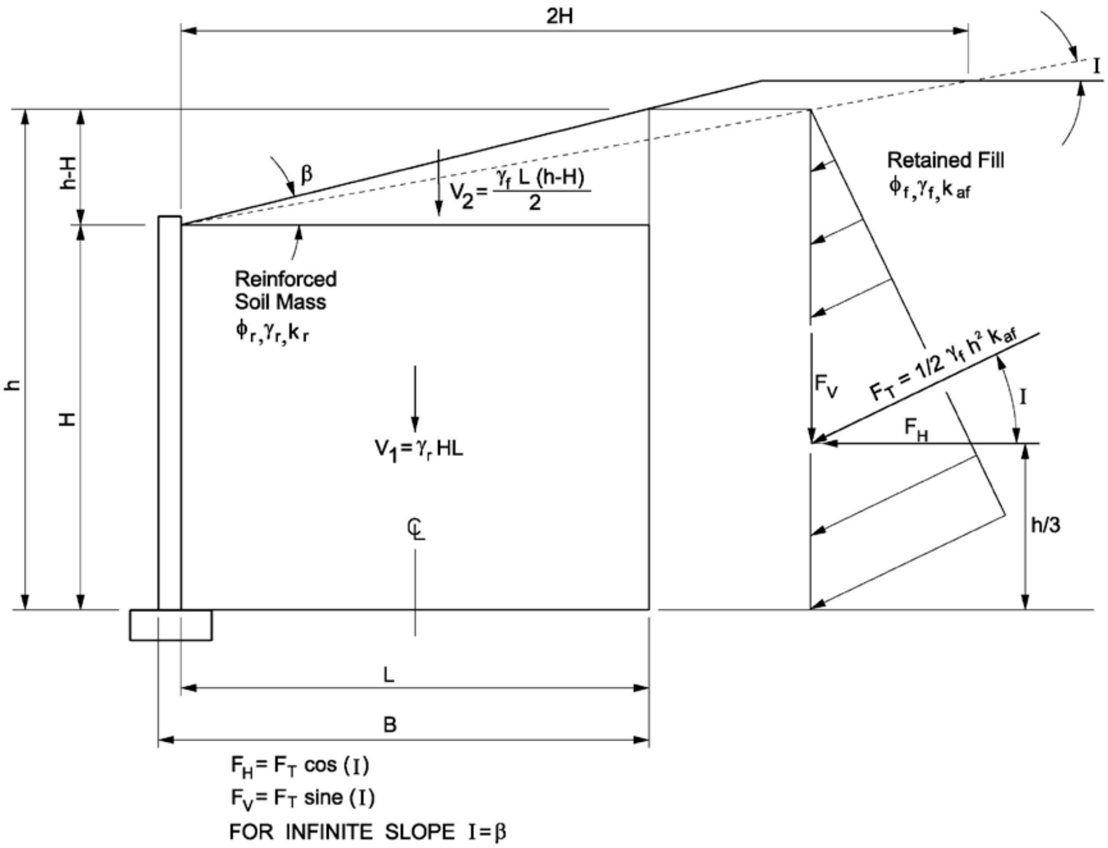


Figure 14.4-4
MSE Walls Earth Pressure for Broken Backfill
(Source *AASHTO LRFD*)

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.

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

When designing a "Modular Block Gravity Wall" with setback, the active earth pressure coefficient K_a shall be determined from the following Coulomb formula. The interface friction angle between the blocks and soil behind the blocks is assumed to be zero.

$$K_a = \frac{\cos^2(\phi_f + A)}{\cos^2 A \cos A (1 + (Z/Y)^{1/2})^2}$$

Where:

Z = $\sin \phi_f \sin(\phi_r - \beta)$

Y = $\cos A \cos(A + \beta)$

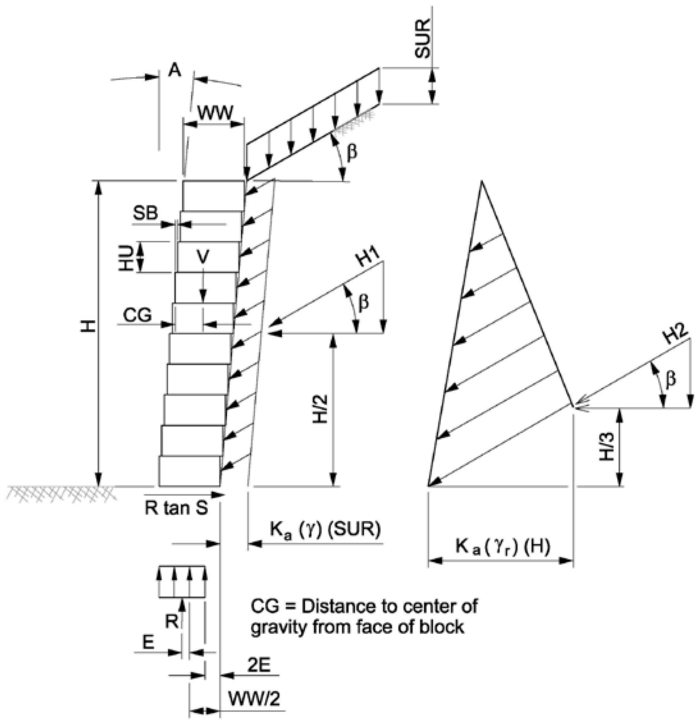


Figure 14.4-5
Modular Block Gravity Wall Analysis

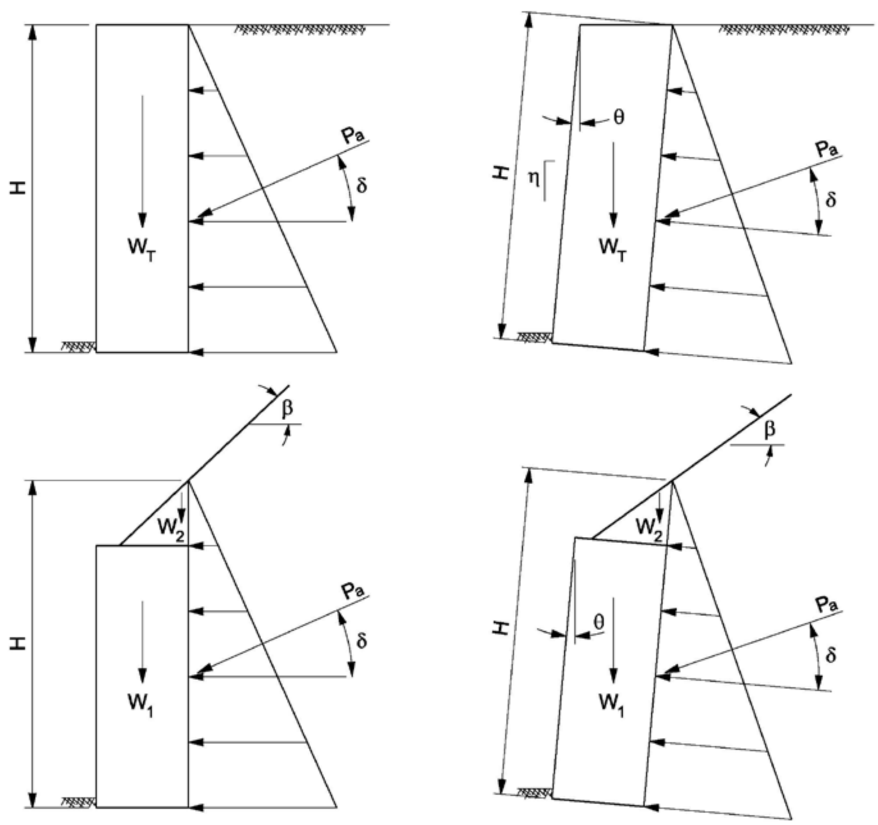
No live load traffic and live load surcharge shall be allowed on modular block gravity walls although they are designed for a minimum live load of 100psf. The density of the blocks is assumed to be 135 pcf and the drainage aggregate inside or between the blocks 120 pcf. The forces acting on a modular block gravity wall are shown in Figure 14.4-5.

Prefabricated Modular Walls

Active earth pressure shall be determined by multiplying vertical loads by the coefficient of active earth pressure (K_a) and using Coulomb earth pressure theory in accordance with **LRFD [3.11.5.3] and LRFD [3.11.5.9]**. See Figure 14.4-6 for earth pressure diagram.

When the rear of the modules form an irregular surface (stepped surface), pressures shall be computed on an average plane surface drawn from the lower back heel of the lowest module as shown in Figure 14.4-7

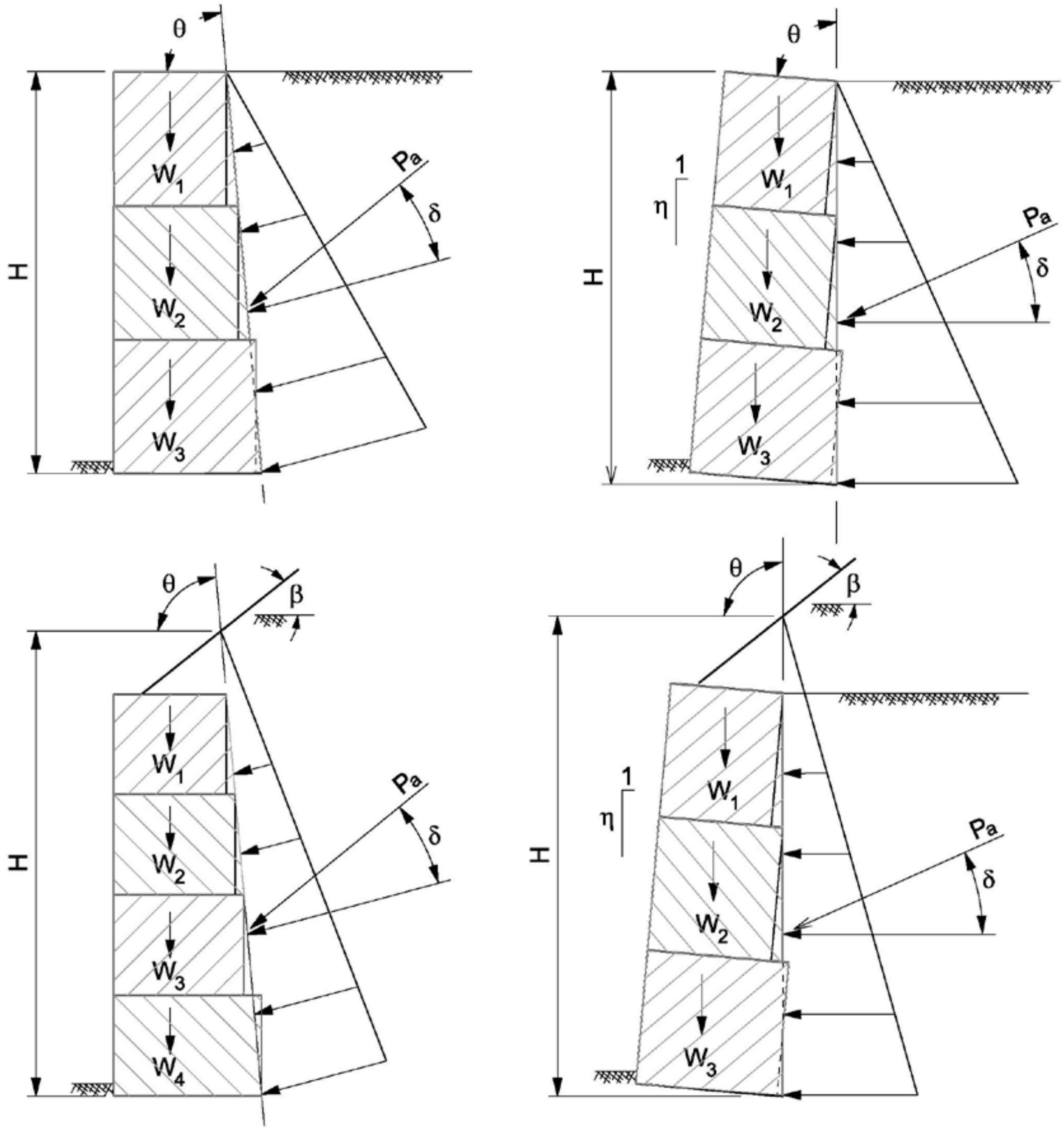
Effect of the backslope soil surcharge and any other surcharge load imposed by existing structure should be accounted as discussed in 14.4.5.4. Trial wedge or Culmann method may also be used to compute the lateral earth pressure as presented in the *Foundation Analysis and Design*, 5th Edition (J. Bowles, 1996).



P_a @ H/3
$P_a = ½\gamma H^2 K_a$

$$K_a = \frac{\cos^2(\phi+\theta)}{\cos^2\theta\,\cos(\theta-\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\,\sin(\phi-\beta)}{\cos(\theta-\delta)\,\cos(\theta+\beta)}}\right]^2}$$

Figure 14.4-6
Lateral Earth Pressure on Concrete Modular Systems of Constant Width
(Source *AASHTO LRFD*)



$$K_a = \frac{\sin^2(\theta+\phi)}{\sin^2\theta\sin(\theta-\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\theta-\delta)\sin(\theta+\beta)}}\right]^2}$$

Figure 14.4-7
Lateral Earth Pressure on Concrete Modular Systems of Variable Width
(Source *AASHTO LRFD*)

14.14 Contract Plan Requirements

The following minimum information shall be required on the plans.

1. Finish grades at rear and front of wall at 25 foot intervals or less.
2. Final cross sections as required for wall designer.
3. Beginning and end stations of wall and offsets from reference line to front face of walls. If reference line is a horizontal curve give offsets from a tangent to the curve.
4. Location of right-of-way boundaries, and construction easements relative to the front face of the walls.
5. Location of utilities if any and indicate whether to remain in place or be relocated or abandoned.
6. Special requirements on top of wall such as copings, railings, or traffic barriers.
7. Footing or leveling pad elevations if different than standard.
8. General notes on standard insert sheets.
9. Soil design parameters for retained soil, backfill soil and foundation soil including angle of internal friction, cohesion, coefficient of sliding friction, groundwater information and ultimate and/or allowable bearing capacity for foundation soil. If piles are required, give skin friction values and end bearing values for displacement piles and/or the allowable steel stress and anticipated driving elevation for end bearing piles.
10. Soil borings.
11. Details of special architectural treatment required for each wall system.
12. Wall systems, system or sub-systems allowed on projects.
13. Abutment details if wall is component of an abutment.
14. Connection and/or joint details where wall joins another structure.
15. Groundwater elevations.
16. Drainage provisions at heel of wall foundations.
17. Drainage at top of wall to divert run-off water.

14.15 Construction Documents

14.15.1 Bid Items and Method of Measurement

Proprietary retaining walls shall include all required bid items necessary to build the wall system provided by the contractor. The unit of measurement shall be square feet and shall include the exposed wall area between the footing and the top of the wall measured to the top of any copings. For setback walls the area shall be based on the walls projection on a vertical plane. The bid item includes designing the walls preparing plans, furnishing and placing all materials, including all excavations, temporary bracing, piling, (including delivered and driven), poured in place or precast concrete or blocks, leveling pads, soil reinforcement systems, structural steel, reinforcing steel, backfills and infills, drainage systems and aggregate, geotextiles, architectural treatment including painting and/or staining, drilled shafts, wall toppings unless excluded by contract, wall plantings, joint fillers, and all labor, tools, equipment and incidentals necessary to complete the work.

The contractor will be paid for the plan quantity as shown on the plans. (The intent is a lump sum bid item but is bid as square feet of wall). The top of wall coping is any type of cap placed on the wall. It does not include any barriers. Measurement is to the bottom of the barrier when computing exposed wall area.

Non-proprietary retaining walls are bid based on the quantity of materials used to construct the wall such as concrete, reinforcing steel, piling, etc. These walls are:

- Cast-in-Place Concrete Cantilever Walls
- Post-and-Panel Walls
- Steel Sheet Piling Walls

14.15.2 Special Provisions

The Structures Design Section has Standard Special Provisions for:

- Wall Modular Block Gravity LRFD, Item SPV.0165.
- Wall Modular Block Mechanically Stabilized Earth LRFD, Item SPV.0165.
- Wall Concrete Panel Mechanically Stabilized Earth LRFD, Item SPV.0165.
- Wall CIP Facing Mechanically Stabilized Earth LRFD, Item SPV.0165.
- Wall Wire Faced Mechanically Stabilized Earth LRFD, Item SPV.0165.
- *Wall Gabion LRFD, SPV under development.*
- *Wall Modular Bin or Crib LRFD, SPV under development.*

- *Temporary Wall Wire Faced Mechanically Stabilized Earth LRFD, SPV under development.*

The designer determines what wall systems(s) are applicable for the project. The approved names of suppliers are inserted for each eligible wall system. The list of approved proprietary wall suppliers is maintained by the Structures Development Section which is responsible for the Approval Process for earth retaining walls, 14.16.

14.16 Submittal Requirements for Pre-Approval Process

14.16.1 General

The following four wall systems require the supplier or manufacturer to submit to the Structural Design Section a package that addresses the items specified in paragraph C.

1. Modular Block Gravity Walls
2. MSE Walls with Modular Block Facings
3. MSE Walls with Precast Concrete Panel Facings
4. Modular Concrete Bin or Crib Walls

14.16.2 General Requirements

Approval of retaining wall systems allows for use of these systems on Wisconsin Department of Transportation (WisDOT) projects upon the manufacturer's certification that the system as furnished to the contractor (or purchasing agency) complies with the design procedures specified in the *Bridge Manual*. WisDOT projects include: State, County and Municipal Federal Aid and authorized County and Municipal State Aid projects in addition to materials purchased directly by the state.

The manufacturer shall perform all specification tests with qualified personnel and maintain an acceptable quality control program. The manufacturer shall maintain records of all its control testing performed in the production of retaining wall systems. These test records shall be available at all times for examination by the Construction Materials Engineer for Highways or designee. Approval of materials will be contingent upon satisfactory compliance with procedures and material conformance to requirements as verified by source and field samples. Sampling will be performed by personnel during the manufacture of project specific materials.

14.16.3 Qualifying Data Required For Approval

Applicants requesting Approval for a specific system shall provide three copies of the documentation showing that they comply with *AASHTO LRFD* and *WisDOT Standard Specifications* and the design criteria specified in the *Bridge Manual*.

1. An overview of the system, including system theory.
2. Laboratory and field data supporting the theory.
3. Detailed design procedures, including sample calculations for installations with no surcharge, level surcharge and sloping surcharge.
4. Details of wall elements, analysis of structural elements, capacity demand ratio, load and resistance factors, estimated life, corrosion design procedure for soil

14.17 References

1. State of Wisconsin, Department of Transportation, *Facilities Development Manual*

2. American Association of State highway and Transportation officials. *Standard Specification for highway Bridges*

3. American Association of State highway and Transportation officials. *AASHTO LRFD Bridge Design Specifications*

4. AASHTO LRFD Bridge Design Specification 4th Edition, 2007, AASHTO, 444 North Capitol Street, N.W., Suite 249, Washington, D.C. 20001.

5. Berg, Christopher and Samtani. *Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes*. Publication No.FHWA-NHI-10-024.2009.

6. Bowles, Joseph E*. Foundation Analysis and Design 4th Edition.* McGraw Hill *1989*

7. Cudoto, Donald P. *Foundation Design Principles and Practices (* 2nd Edition*),* Prentice Halls

8. National Concrete Masonry Association, "Design Manual for Segmental Retaining Walls", 2302 Horse Pen Road, Herndon, Virginia 22071-3406.

9. Lazarte, Elias, Espinoza, Sabatini. *Geotechnical Engineering Circular No 7. Soil Nailing Walls*, FHWA

10. Publication No FHWA-SA-96-069R , "*Manual for design and construction of Soil Nail walls*

11. Publication No.FHWA-HI-98-032, "Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures".

12. Publication No.FHWA-NHI-07-071, "Earth retaining Structures".

13. Publication No.FHWA-NHI-09-083, "Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures".

14. Publication No. FHWA-NHI-09-087, "Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced slopes"

15. Publication No.FHWA-NHI-10-024, "Design and Construction of Mechanically Stabilized earth Walls and Reinforced Soil Slopes-Volume I".

16. Publication No.FHWA-NHI-10-025, "Design and Construction of Mechanically Stabilized earth Walls and Reinforced Soil Slopes-Volume II".

14.18 Design Examples

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

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

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

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

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

$\beta = 2.0$

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

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

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

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

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

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

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

E14-1.7.1.2 Evaluate Heel Flexural Strength

$V_u = 21.9$ kip/ft

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

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

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

Calculate the flexural resistance factor ϕ_F:

$\beta_1 = 0.85$

$c = \frac{a}{\beta_1}$ $\boxed{c = 1.49}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \frac{d_s}{c} < \frac{5}{3} \\ 0.65 + 0.15\left(\frac{d_s}{c} - 1\right) & \text{if } \frac{5}{3} \le \frac{d_s}{c} \le \frac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$

$\boxed{\phi_F = 0.90}$

based on f_y = 60 ksi, **LRFD [5.5.4.2.1], [Table C5.7.2.1-1]**

Note: if $\phi_F = 0.75$ Section is compression-controlled

if $0.75 < \phi_F < 0.90$ Section is in transition

if $\phi_F = 0.90$ Section is tension-controlled

Calculate the flexural factored resistance, M_r:

$$M_r = \phi_F\, M_n$$

$\boxed{M_r = 71.2}$ kip-ft/ft

$\boxed{M_u = 47.9}$ kip-ft/ft

Is M_u less than M_r?

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

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

$$f_r = 0.24\sqrt{f'_c}$$

$\boxed{f_r = 0.449}$ ksi

$$I_g = \frac{1}{12}\, b\,(D\,12)^3$$

$\boxed{I_g = 13824}$ in^4

$$y_t = \frac{1}{2}\, D\, 12$$

$\boxed{y_t = 12.00}$ in

$$S_c = \frac{I_g}{y_t}$$

$\boxed{S_c = 1152}$ in^3

$$M_{cr} = \gamma_3\left(\gamma_1\, f_r\right) S_c \quad \text{therefore,} \quad M_{cr} = 1.1\, f_r\, S_c$$

Where:

$\gamma_1 = 1.6$ flexural cracking variability factor

$\gamma_3 = 0.67$ ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement for A615, Grade 60 reinforcement

$$M_{cr} = 1.1\, f_r\, S_c\, \frac{1}{12}$$

$\boxed{M_{cr} = 47.4}$ kip-ft/ft

E14-1.7.2.2 Evaluate Toe Flexural Strength

$V_u = 13.2$ kip/ft

$M_u = V_u \frac{A}{2}$ $\boxed{M_u = 23.2}$ kip-ft/ft

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

$M_n = A_s f_y \left(d_s - \frac{a}{2}\right) \frac{1}{12}$ $\boxed{M_n = 42.0}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$\beta_1 = 0.85$

$c = \frac{a}{\beta_1}$ $\boxed{c = 0.82}$ in

$$\phi_F = \begin{vmatrix} 0.75 \text{ if } \frac{d_s}{c} < \frac{5}{3} \\ 0.65 + 0.15\left(\frac{d_s}{c} - 1\right) \text{ if } \frac{5}{3} \le \frac{d_s}{c} \le \frac{8}{3} \\ 0.90 \text{ otherwise} \end{vmatrix}$$

$\boxed{\phi_F = 0.90}$

based on f_y = 60 ksi, **LRFD [5.5.4.2.1], [Table C5.7.2.1-1]**

Calculate the flexural factored resistance, M_r:

$M_r = \phi_F M_n$ $\boxed{M_r = 37.8}$ kip-ft/ft

Is M_u less than M_r? $\boxed{\text{check} = \text{"OK"}}$

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

$f_r = 0.24 \sqrt{f'_c}$ $\boxed{f_r = 0.449}$ ksi

$I_g = \frac{1}{12} b (D\ 12)^3$ $\boxed{I_g = 13824}$ in^4

$y_t = \frac{1}{2} D\ 12$ $\boxed{y_t = 12.00}$ in

$S_c = \frac{I_g}{y_t}$ $\boxed{S_c = 1152}$ in^3

$M_{cr} = 1.1 f_r S_c \frac{1}{12}$ from E14-1.7.1.2 $\boxed{M_{cr} = 47.4}$ kip-ft/ft

$\boxed{1.33\ M_u = 30.8}$ kip-ft/ft

Is M_r greater than the lesser value of M_{cr} and $1.33*M_u$?

$\boxed{check = "OK"}$

E14-1.7.3 Evaluate Stem Strength

Unfactored Stem Horizontal Loads and Moments:

$$H_1 = \gamma_f\ h_{eq}\ (h' - t)\ k_a\ \cos(90\ deg - \theta + \delta)$$ $\boxed{H_1 = 1.2}$ kip/ft

$$H_2 = \frac{1}{2}\ \gamma_f\ (h' - t)^2\ k_a\ \cos(90\ deg - \theta + \delta)$$ $\boxed{H_2 = 5.0}$ kip/ft

$$M_1 = H_1 \left(\frac{h' - t}{2}\right)$$ $\boxed{M_1 = 10.0}$ kip-ft/ft

$$M_2 = H_2 \left(\frac{h' - t}{3}\right)$$ $\boxed{M_2 = 28.4}$ kip-ft/ft

Factored Stem Horizontal Loads and Moments:

for **Strength Ib**:

$$H_{u1} = 1.75\ H_1 + 1.50\ H_2$$ $\boxed{H_{u1} = 9.6}$ kip/ft

$$M_{u1} = 1.75\ M_1 + 1.50\ M_2$$ $\boxed{M_{u1} = 60.0}$ kip-ft/ft

for **Service I**:

$$H_{u3} = 1.00\ H_1 + 1.00\ H_2$$ $\boxed{H_{u3} = 6.2}$ kip/ft

$$M_{u3} = 1.00\ M_1 + 1.00\ M_2$$ $\boxed{M_{u3} = 38.4}$ kip-ft/ft

E14-1.7.3.1 Evaluate Stem Shear Strength at Footing

$$V_u = H_{u1}$$ $\boxed{V_u = 9.6}$ kip/ft

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.8.3.3]**

$V_{n1} = V_c$ **LRFD [Eq 5.8.3.3-1]**

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

$V_{n2} = 0.25\ f'_c\ b_v\ d_v$ **LRFD [Eq 5.8.3.3-2]**

Compute the shear resistance due to concrete, V_c :

$cover = 2.0$ in

$s = 10.0$ in (bar spacing)

$Bar_{No} = 8$ (transverse bar size)

$Bar_D = 1.00$ in (transverse bar diameter)

$Bar_A = 0.79$ in^2 (transverse bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}} \qquad \boxed{A_s = 0.95} \; in^2/ft$$

$$d_s = T_b\, 12 - cover - \frac{Bar_D}{2} \qquad \boxed{d_s = 23.0} \; in$$

$$a = \frac{A_s\, f_y}{0.85\, f'_c\, b} \qquad \boxed{a = 1.6} \; in$$

$$d_{v1} = d_s - \frac{a}{2} \qquad \boxed{d_{v1} = 22.2} \; in$$

$$d_{v2} = 0.9\, d_s \qquad \boxed{d_{v2} = 20.7} \; in$$

$$d_{v3} = 0.72\, T_b\, 12 \qquad \boxed{d_{v3} = 18.4} \; in$$

$$d_v = \max(d_{v1}, d_{v2}, d_{v3}) \qquad \boxed{d_v = 22.2} \; in$$

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

$\beta = 2.0$

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

$$V_{n1} = V_c \qquad \boxed{V_{n1} = 31.5} \; kip/ft$$

$$V_{n2} = 0.25\, f'_c\, b\, d_v \qquad \boxed{V_{n2} = 233.1} \; kip/ft$$

$$V_n = \min(V_{n1}, V_{n2}) \qquad \boxed{V_n = 31.5} \; kip/ft$$

$$V_r = \phi_v\, V_n \qquad \boxed{V_r = 28.4} \; kip/ft$$

$$\boxed{V_u = 9.6} \; kip/ft$$

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

E14-1.7.3.2 Evaluate Stem Flexural Strength at Footing

$M_u = M_{u1}$ $\boxed{M_u = 60.0}$ kip-ft/ft

Calculate the capacity of the stem in flexure at the face of the footing:

$$M_n = A_s\, f_y \left(d_s - \frac{a}{2}\right)\frac{1}{12}$$ $\boxed{M_n = 105.2}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$\beta_1 = 0.85$

$$c = \frac{a}{\beta_1}$$ $\boxed{c = 1.87}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \dfrac{d_s}{c} < \dfrac{5}{3} \\ 0.65 + 0.15\left(\dfrac{d_s}{c} - 1\right) & \text{if } \dfrac{5}{3} \le \dfrac{d_s}{c} \le \dfrac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$

$\boxed{\phi_F = 0.90}$

based on f_y = 60 ksi, **LRFD [5.5.4.2.1], [Table C5.7.2.1-1]**

Calculate the flexural factored resistance, M_r:

$M_r = \phi_F\, M_n$ $\boxed{M_r = 94.7}$ kip-ft/ft

$\boxed{M_u = 60.0}$ kip-ft/ft

Is M_u less than M_r? $\boxed{check = "OK"}$

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

$f_r = 0.24\sqrt{f'_c}$ $\boxed{f_r = 0.45}$ ksi

$$I_g = \frac{1}{12}\, b\, (T_b\, 12)^3$$ $\boxed{I_g = 16581}$ in^4

$$y_t = \frac{1}{2}\, T_b\, 12$$ $\boxed{y_t = 12.8}$ in

$$S_c = \frac{I_g}{y_t}$$ $\boxed{S_c = 1301}$ in^3

$M_{cr\_s} = 1.1\ f_r\ S_c\ \frac{1}{12}$ from E14-1.7.1.2 $\boxed{M_{cr\_s} = 53.5}$ kip-ft/ft

$\boxed{1.33\ M_u = 79.9}$ kip-ft/ft

Is M_r greater than the lesser value of M_{cr} and $1.33*M_u$? $\boxed{\text{check} = \text{"OK"}}$

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

$\rho = \frac{A_s}{d_s\ b}$ $\boxed{\rho = 0.00343}$

$n = \frac{E_s}{E_c}$ $\boxed{n = 8.09}$

$k = \sqrt{(\rho\ n)^2 + 2\ \rho\ n} - \rho\ n$ $\boxed{k = 0.210}$

$j = 1 - \frac{k}{3}$ $\boxed{j = 0.930}$

$d_c = \text{cover} + \frac{Bar_D}{2}$ $\boxed{d_c = 2.5}$ in

$f_{ss} = \frac{M_{u3}}{A_s\ j\ d_s}\ 12 \le 0.6\ f_y$ $\boxed{f_{ss} = 22.7}$ ksi $\le 0.6\ f_y$ O.K.

$h = T_b\ 12$

$\beta_s = 1 + \frac{d_c}{0.7\left(h - d_c\right)}$ $\boxed{\beta_s = 1.2}$

$\gamma_e = 1.0$ for Class 1 exposure

$s_{max} = \frac{700\ \gamma_e}{\beta_s\ f_{ss}} - 2\ d_c$ $\boxed{s_{max} = 21.7}$ in

$\boxed{s = 10.0}$ in

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

E14-1.7.3.3 Transfer of Force at Base of Stem

Specification requires that the transfer of lateral forces from the stem to the footing be in accordance with the shear-transfer provisions of **LRFD [5.8.4]**. That calculation will not be presented. Refer to E13-1.9.3 for a similar computation.

E14-1.7.4 Temperature and Shrinkage Steel

Look at temperature and shrikage requirements

E14-1.7.4.1 Temperature and Shrinkage Steel for Footing

The footing will not be exposed to daily temperature changes. Thus temperature and shrinkage steel is not required. However, #4 bars at 18" o.c. (max) are placed longitudinally to serve as spacers.

E14-1.7.4.2 Temperature and Shrinkage Steel of Stem

The stem will be exposed to daily temperature changes. In accordance with **LRFD [5.10.8** the stem shall provide temperature and shrinkage steel on each face and in each direction as calculated below:

$s = 18.0$ in (bar spacing)

$Bar_{No} = 4$ (bar size)

$Bar_D = 0.50$ in (temperature and shrinkage bar diameter)

$Bar_A = 0.20$ in^2 (temperature and shrinkage bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}}$$ (temperature and shrinkage provided)

$\boxed{A_s = 0.13}$ in^2/ft

$b_s = (H - D)\ 12$ least width of stem $\boxed{b_s = 216.0}$ in

$h_s = T_t\ 12$ least thickness of stem $\boxed{h_s = 12.0}$ in

$$A_{ts} = \frac{1.3\ b_s\ h_s}{2\left(b_s + h_s\right)\ f_y}$$ Area of reinforcement per foot, on each face and in each direction $\boxed{A_{ts} = 0.12}$ in^2/ft

Is $0.11 \le A_s \le 0.60$? $\boxed{check = \text{"OK"}}$

Is $A_s > A_{ts}$? $\boxed{check = \text{"OK"}}$

Determine the location of the pile critical perimeter. Assume that the critical section is outside of the footing and only include the portion of the shear perimeter is located within the footing:

$$b_{o\_xx} = 1.25\ 12 + \frac{B_{xx}}{2} + \frac{d_{v\_toe}}{2}$$ $\boxed{b_{o\_xx} = 32.5}$ in

$$b_{o\_yy} = 1.25\ 12 + \frac{B_{yy}}{2} + \frac{d_{v\_toe}}{2}$$ $\boxed{b_{o\_yy} = 32.3}$ in

$$\beta_{c\_pile} = \frac{b_{o\_xx}}{b_{o\_yy}}$$ $\boxed{\beta_{c\_pile} = 1.004}$ in

$$b_{o\_pile} = b_{o\_xx} + b_{o\_yy}$$ $\boxed{b_{o\_pile} = 64.8}$ in

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.13.3.6.3]**

$$V_{n1} = \left(0.063 + \frac{0.126}{\beta_{c\_pile}}\right)\sqrt{f'_c}\, b_{o\_pile}\, d_{v\_toe}$$ $\boxed{V_{n1} = 523.1}$ kip/ft

$$V_{n2} = 0.126\sqrt{f'_c}\, b_{o\_pile}\, d_{v\_toe}$$ $\boxed{V_{n2} = 349.7}$ kip/ft

$$V_n = \min(V_{n1}, V_{n2})$$ $\boxed{V_n = 349.7}$ kip/ft

$$V_r = \phi_v V_n$$ $\boxed{V_r = 314.7}$ kip/ft

$\boxed{V_u = 150.2}$ kip/ft

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

E14-4.7.1.3 Evaluate Top Transverse Reinforcement Strength

Top transverse reinforcement strength is determined by assuming the heel acts as a cantilever member supporting its own weight and loads acting above it. Pile reactions may be used to decrease this load.

For **Strength Ib**:

$$V_u = 1.25\left(\frac{C}{B} V_4\right) + 1.35\left(V_7 + V_8 + V_9\right) + 1.75\left(V_{10}\right) + 1.50\left(V_{11}\right)$$

$\boxed{V_u = 27.0}$ kip/ft

$$M_u = V_u \frac{C}{2}$$ $\boxed{M_u = 66.3}$ kip-ft/ft

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

$$M_n = A_{s\_heel} f_y \left(d_{s\_heel} - \frac{a\_heel}{2}\right)\frac{1}{12}$$ $\boxed{M_n = 107.6}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$\beta_1 = 0.85$

$c = \dfrac{a\_heel}{\beta_1}$ $\boxed{c = 1.58}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \dfrac{d_{s}\_heel}{c} < \dfrac{5}{3} \\ 0.65 + 0.15\left(\dfrac{d_{s}\_heel}{c} - 1\right) & \text{if } \dfrac{5}{3} \le \dfrac{d_{s}\_heel}{c} \le \dfrac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$

$\boxed{\phi_F = 0.90}$

based on f_y = 60 ksi, **LRFD [5.5.4.2.1], [Table C5.7.2.1-1]**

Note: if $\phi_F = 0.75$ Section is compression-controlled
if $0.75 < \phi_F < 0.90$ Section is in transition
if $\phi_F = 0.90$ Section is tension-controlled

Calculate the flexural factored resistance, M_r:

$M_r = \phi_F M_n$ $\boxed{M_r = 96.8}$ kip-ft/ft

$\boxed{M_u = 66.3}$ kip-ft/ft

Is M_u less than M_r? $\boxed{check = \text{"OK"}}$

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

$f_r = 0.24\sqrt{f'_c}$ $\boxed{f_r = 0.449}$ ksi

$I_g = \dfrac{1}{12} b (D\ 12)^3$ $\boxed{I_g = 27000}$ in^4

$y_t = \dfrac{1}{2} D\ 12$ $\boxed{y_t = 15.00}$ in

$S_c = \dfrac{I_g}{y_t}$ $\boxed{S_c = 1800}$ in^3

$M_{cr} = \gamma_3(\gamma_1 f_r) S_c$ therefore, $M_{cr} = 1.1 f_r S_c$

Where:

$\gamma_1 = 1.6$ flexural cracking variability factor

$\gamma_3 = 0.67$ ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement for A615, Grade 60 reinforcement

$M_{cr} = 1.1\, f_r\, S_c\, \frac{1}{12}$ | $\boxed{M_{cr} = 74.1}$ kip-ft/ft

$\boxed{1.33\, M_u = 88.2}$ kip-ft/ft

Is M_r greater than the lesser value of M_{cr} and 1.33*M_u? | $\boxed{check = \text{"OK"}}$

E14-4.7.1.4 Evaluate Bottom Transverse Reinforcement Strength

Bottom transverse reinforcement strength is determined by using the maximum pile reaction.

Determine the moment arms

$arm\_v1 = A - y_{p1}$ | $\boxed{arm\_v1 = 3.5}$ ft

$arm\_v2 = A - y_{p2}$ | $\boxed{arm\_v2 = 0.8}$ ft

Determine the moment for **Strength Ia**:

$V_{u\_1a} = P_{U1a}\, NP_1$ | $\boxed{V_{u\_1a} = 9.8}$ kip/ft

$V_{u\_2a} = P_{U2a}\, NP_2$ | $\boxed{V_{u\_2a} = 9.5}$ kip/ft

$M_{u\_Ia} = V_{u\_1a}\, arm\_v1 + V_{u\_2a}\, arm\_v2$ | $\boxed{M_{u\_Ia} = 41.6}$ kip-ft/ft

Determine the moment for **Strength Ib**:

$V_{u\_1b} = P_{U1b}\, NP_1$ | $\boxed{V_{u\_1b} = 6.8}$ kip/ft

$V_{u\_2b} = P_{U2b}\, NP_2$ | $\boxed{V_{u\_2b} = 12.5}$ kip/ft

$M_{u\_Ib} = V_{u\_1b}\, arm\_v1 + V_{u\_2b}\, arm\_v2$ | $\boxed{M_{u\_Ib} = 33.3}$ kip-ft/ft

Determine the design moment:

$M_u = \max\left(M_{u\_Ia}, M_{u\_Ib}\right)$ | $\boxed{M_u = 41.6}$ kip-ft/ft

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

$M_n = A_{s\_toe}\, f_y \left(d_{s\_toe} - \frac{a\_toe}{2}\right) \frac{1}{12}$ | $\boxed{M_n = 91.6}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$\beta_1 = 0.85$

$c = \frac{a\_toe}{\beta_1}$ | $\boxed{c = 1.58}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \dfrac{d_{s\_toe}}{c} < \dfrac{5}{3} \\ 0.65 + 0.15\left(\dfrac{d_{s\_toe}}{c} - 1\right) & \text{if } \dfrac{5}{3} \le \dfrac{d_{s\_toe}}{c} \le \dfrac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$

$\boxed{\phi_F = 0.90}$

based on f_y = 60 ksi,
LRFD [5.5.4.2.1], [Table C5.7.2.1-1]

Calculate the flexural factored resistance, M_r:

$M_r = \phi_F\, M_n$ $\boxed{M_r = 82.4}$ kip-ft/ft

$\boxed{M_u = 41.6}$ kip-ft/ft

Is M_u less than M_r? $\boxed{\text{check} = \text{"OK"}}$

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

$f_r = 0.24\sqrt{f'_c}$ $\boxed{f_r = 0.449}$ ksi

$I_g = \dfrac{1}{12}\, b\,(D\ 12)^3$ $\boxed{I_g = 27000}$ in^4

$y_t = \dfrac{1}{2}\, D\ 12$ $\boxed{y_t = 15.00}$ in

$S_c = \dfrac{I_g}{y_t}$ $\boxed{S_c = 1800}$ in^3

$M_{cr} = 1.1\, f_r\, S_c\, \dfrac{1}{12}$ from E14-4.7.1.3 $\boxed{M_{cr} = 74.1}$ kip-ft/ft

$\boxed{1.33\, M_u = 55.3}$ kip-ft/ft

Is M_r greater than the lesser value of M_{cr} and $1.33*M_u$? $\boxed{\text{check} = \text{"OK"}}$

E14-4.7.1.5 Evaluate Longitudinal Reinforcement Strength

The structural design of the longitudinal reinforcement, assuming the footing acts as a continuous beam over pile supports, is calculated using the maximum pile reactions.

Compute the effective shear depth, d_v ,for the longitudinal reinforcement

$cover = 6.0$ in

$s = 12.0$ in (bar spacing)

$Bar_{No} = 5$ (longitudinal bar size)

$Bar_D = 0.625$ in (longitudinal bar diameter)

$Bar_A = 0.310$ in² (longitudinal bar area)

$$A_{s\_long} = \frac{Bar_A}{\frac{s}{12}} \qquad \boxed{A_{s\_long} = 0.31} \text{ in}^2\text{/ft}$$

$$d_s = D\ 12 - cover - Bar_{D\_toe} - \frac{Bar_D}{2} \qquad \boxed{d_s = 22.8} \text{ in}$$

$$a\_long = \frac{A_{s\_long}\ f_y}{0.85\ f'_c\ b} \qquad \boxed{a\_long = 0.5} \text{ in}$$

$$d_{v1} = d_s - \frac{a\_long}{2} \qquad \boxed{d_{v1} = 22.6} \text{ in}$$

$$d_{v2} = 0.9\ d_s \qquad \boxed{d_{v2} = 20.5} \text{ in}$$

$$d_{v3} = 0.72\ D\ 12 \qquad \boxed{d_{v3} = 21.6} \text{ in}$$

$$d_{v\_long} = \max(d_{v1}, d_{v2}, d_{v3}) \qquad \boxed{d_{v\_long} = 22.6} \text{ in}$$

Calculate the design moment using a uniform vertical load:

$$L_{pile} = \max(P_1, P_2, P_3) \qquad \boxed{L_{pile} = 8.0} \text{ ft}$$

$$w_u = \frac{V\_Ib}{B} \qquad \boxed{w_u = 3.2} \text{ kip/ft/ft}$$

$$M_u = \frac{w_u\ L_{pile}^2}{10} \qquad \boxed{M_u = 20.3} \text{ kip-ft/ft}$$

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

$$M_n = A_{s\_long}\, f_y \left(d_s - \frac{a\_long}{2}\right)\frac{1}{12}$$ $\boxed{M_n = 35.0}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$$\beta_1 = 0.85$$

$$c = \frac{a\_toe}{\beta_1}$$ $\boxed{c = 1.58}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \dfrac{d_s}{c} < \dfrac{5}{3} \\ 0.65 + 0.15\left(\dfrac{d_s}{c} - 1\right) & \text{if } \dfrac{5}{3} \le \dfrac{d_s}{c} \le \dfrac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$

$\boxed{\phi_F = 0.90}$

based on f_y = 60 ksi,
LRFD [5.5.4.2.1], [Table C5.7.2.1-1]

Calculate the flexural factored resistance, M_r:

$$M_r = \phi_F\, M_n$$ $\boxed{M_r = 31.5}$ kip-ft/ft

Is M_u less than M_r? $\boxed{check = \text{"OK"}}$

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

$$f_r = 0.24\sqrt{f'_c}$$ $\boxed{f_r = 0.449}$ ksi

$$I_g = \frac{1}{12}\, b\,(D\ 12)^3$$ $\boxed{I_g = 27000}$ in^4

$$y_t = \frac{1}{2}\, D\ 12$$ $\boxed{y_t = 15.00}$ in

$$S_c = \frac{I_g}{y_t}$$ $\boxed{S_c = 1800}$ in^3

$$M_{cr} = 1.1\, f_r\, S_c\, \frac{1}{12}$$ from E14-4.7.1.3 $\boxed{M_{cr} = 74.1}$ kip-ft/ft

$\boxed{1.33\ M_u = 27.1}$ kip-ft/ft

Is M_r greater than the lesser value of M_{cr} and 1.33*M_u? $\boxed{check = \text{"OK"}}$

$\beta_1 = 0.85$

$c = \dfrac{a}{\beta_1}$ $\boxed{c = 1.98}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \dfrac{d_s}{c} < \dfrac{5}{3} \\ 0.65 + 0.15\left(\dfrac{d_s}{c} - 1\right) & \text{if } \dfrac{5}{3} \le \dfrac{d_s}{c} \le \dfrac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$

$\boxed{\phi_F = 0.90}$

based on f_y = 60 ksi,
LRFD [5.5.4.2.1], [Table C5.7.2.1-1]

Calculate the flexural factored resistance, M_r:

$M_r = \phi_F\, M_n$ $\boxed{M_r = 111.2}$ kip-ft/ft

$\boxed{M_u = 94.0}$ kip-ft/ft

Is M_u less than M_r? $\boxed{\text{check} = \text{"OK"}}$

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

$f_r = 0.24\sqrt{f'_c}$ $\boxed{f_r = 0.45}$ ksi

$I_g = \dfrac{1}{12}\, b\, (T_b\, 12)^3$ $\boxed{I_g = 22247}$ in^4

$y_t = \dfrac{1}{2}\, T_b\, 12$ $\boxed{y_t = 14.1}$ in

$S_c = \dfrac{I_g}{y_t}$ $\boxed{S_c = 1582}$ in^3

$M_{cr\_s} = 1.1\, f_r\, S_c\, \dfrac{1}{12}$ from E14-4.7.1.3 $\boxed{M_{cr\_s} = 65.1}$ kip-ft/ft

$\boxed{1.33\, M_u = 125.0}$ kip-ft/ft

Is M_r greater than the lesser value of M_{cr} and 1.33*M_u? $\boxed{\text{check} = \text{"OK"}}$

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

$$\rho = \frac{A_s}{d_s\ b} \qquad \boxed{\rho = 0.00326}$$

$$n = \frac{E_s}{E_c} \qquad \boxed{n = 8.09}$$

$$k = \sqrt{(\rho\ n)^2 + 2\ \rho\ n} - \rho\ n \qquad \boxed{k = 0.205}$$

$$j = 1 - \frac{k}{3} \qquad \boxed{j = 0.932}$$

$$d_c = cover + \frac{Bar_D}{2} \qquad \boxed{d_c = 2.6}\ \text{in}$$

$$f_{ss} = \frac{M_{u3}}{A_s\ j\ d_s}\ 12 \le 0.6\ f_y \qquad \boxed{f_{ss} = 31.0}\ \text{ksi} \quad \le 0.6\ f_y\ \text{O.K.}$$

$$h = T_b\ 12$$

$$\beta_s = 1 + \frac{d_c}{0.7\left(h - d_c\right)} \qquad \boxed{\beta_s = 1.1}$$

$$\gamma_e = 1.00 \qquad \text{for Class 1 exposure}$$

$$s_{max} = \frac{700\ \gamma_e}{\beta_s\ f_{ss}} - 2\ d_c \qquad \boxed{s_{max} = 14.6}\ \text{in}$$

$$\boxed{s = 12.0}\ \text{in}$$

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

E14-4.7.2.3 Transfer of Force at Base of Stem

Specification requires that the transfer of lateral forces from the stem to the footing be in accordance with the shear-transfer provisions of **LRFD [5.8.4]**. That calculation will not be presented. Refer to E13-1.9.3 for a similar computation.

E14-4.7.3 Temperature and Shrinkage Steel

Evaluate temperature and shrinkage requirements

E14-4.7.3.1 Temperature and Shrinkage Steel for Footing

The footing will not be exposed to daily temperature changes. Thus temperature and shrinkage steel is not required.

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The reinforcing bars presented in the "Bar Size and Spacing" column (the third column) in Table 17.5-3 and Table 17.5-4 are for one layer only. Identical steel should be placed in both the top and bottom layers, except for continuity steel.

17.5.3.3 Empirical Design of Slab on Girders

WisDOT policy item:

Approval from the Bureau of Structures Chief Structural Design Engineer is required for use of the empirical design method.

In addition to the traditional design method for decks, as described above, AASHTO also provides specifications for an empirical design method. This method, which is new to *AASHTO LRFD*, does not require the computation of design moments and is simpler to apply than the traditional design method. However, it is applicable only under specified design conditions. The empirical design method should not be used on bridge decks with heavy truck traffic. The empirical design method is described in **LRFD [9.7.2]**.

17.6 Cantilever Slab Design

For deck slabs on girders, the deck overhang must also be designed. Design of the deck overhang involves the following two steps:

1. Design for flexure in deck overhang based on strength and extreme event limit states.

2. Check for cracking in overhang based on service limit state.

The locations of the design sections are illustrated in Figure 17.6-1.

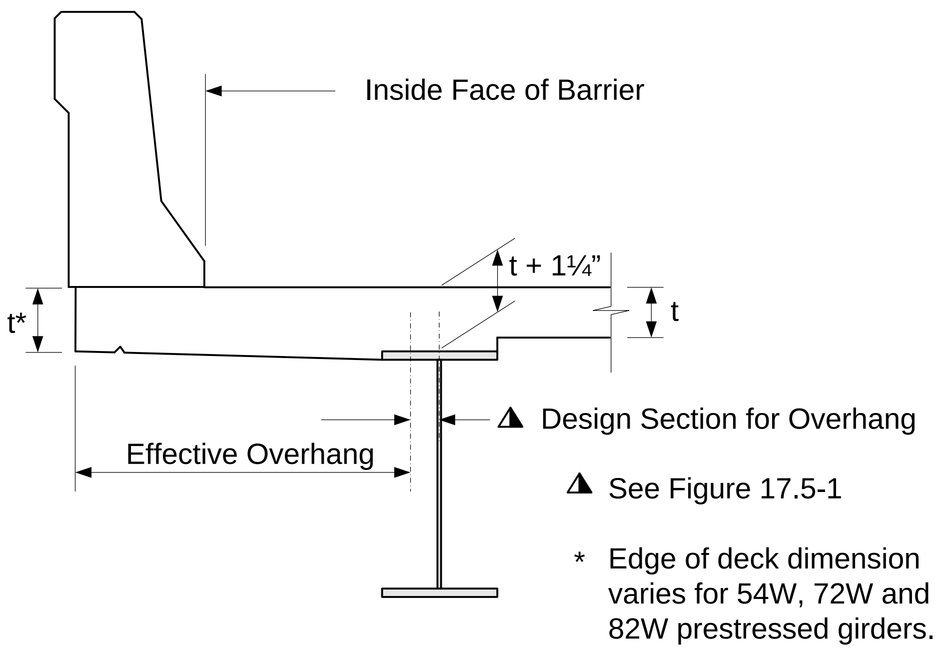

Figure 17.6-1
Deck Overhang Design Section

As described in **LRFD [A13.4]**, deck overhangs must be designed to satisfy three different design cases. These three design cases are summarized in Table 17.6-1.

Design Case	Applied Loads	Limit State	Design Locations
Design Case 1	Horizontal vehicular collision force and dead loads	Extreme Event II	At inside face of barrier At design section for overhang
Design Case 2 (usually does not control)	Vertical vehicular collision force and dead loads	Extreme Event II	At inside face of barrier At design section for overhang

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In regions where stress reversal takes place, continuous concrete slabs will be doubly reinforced. At these locations, the full stress range in the reinforcing bars from tension to compression is considered.

In regions of compressive stress due to permanent loads, fatigue shall be considered only if this compressive stress is less than 1.5 times the maximum tensile live load stress from the fatigue truck. The section properties for fatigue investigations shall be based on cracked sections where the sum of stresses, due to unfactored permanent loads, and 1.5 times the fatigue load is tensile and exceeds $0.095\,(f'_c)^{1/2}$.

The factored stress range, Q, shall be calculated using factored loads described in 18.3.5.1. The factored resistance, R_r , shall be calculated as in 18.3.5.2.1.

Then check that, Q (factored stress range) $\leq R_r$ is satisfied.

Reference is made to the design example in 18.5 of this chapter for computations relating to reinforcement remaining in tension throughout the fatigue cycle, or going through tensile and compressive stresses during the fatigue cycle.

18.4.6.3 Check for Crack Control

Service Limit State considerations and assumptions are detailed in **LRFD [5.5.2, 5.7.1, 5.7.3.4]**.

The area of longitudinal slab reinforcement, A_s , should be checked for crack control at locations where maximum tensile stress occurs along the structure, and for haunched slab structures, checked at the haunch/slab intercepts. The area should also be checked for crack control at bar reinforcement cutoff locations using Service I Limit State. Check the reinforcement in an interior and exterior strip (edge beam).

The use of high-strength steels and the acceptance of design methods where the reinforcement is stressed to higher proportions of the yield strength, makes control of flexural cracking by proper reinforcing details more significant than in the past. The width of flexural cracks is proportional to the level of steel tensile stress, thickness of concrete cover over the bars, and spacing of reinforcement. Improved crack control is obtained when the steel reinforcement is well distributed over the zone of maximum concrete tension.

Crack control criteria shall be applied when the tension in the cross-section exceeds 80% of the modulus of rupture, f_r , specified in **LRFD [5.4.2.6]**, for Service I Limit State. The spacing of reinforcement, s, in the layer closest to the tension face shall satisfy:

$$s \leq (700\,\gamma_e / \beta_s f_{ss}) - 2\,(d_c) \quad \text{(in)}$$

in which:

$$\beta_s = 1 + (d_c) / 0.7\,(h - d_c)$$

Where:

γ_e = 1.00 for Class 1 exposure condition (bottom reinforcement)

γ_e = 0.75 for Class 2 exposure condition (top reinforcement)

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in)

f_{ss} = tensile stress in steel reinforcement (ksi) $\leq 0.6 f_y$; use factored loads described in 18.3.4.1 at the Service I Limit State, to calculate (f_{ss})

h = overall depth of the section (in)

18.4.6.4 Minimum Reinforcement Check

The area of longitudinal slab reinforcement, A_s , should be checked for minimum reinforcement requirement at locations along the structure **LRFD [5.7.3.3.2]**.

The amount of tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r , or moment capacity, at least equal to the lesser of:

M_{cr} (or) 1.33 M_u

$M_{cr} = \gamma_3 (\gamma_1 f_r) S = 1.1 f_r (I_g / c)$; $S = I_g / c$

Where:

f_r = $0.24 (f'c)^{1/2}$ modulus of rupture (ksi) **LRFD [5.4.2.6]**

γ_1 = 1.6 flexural cracking variability factor

γ_3 = 0.67 ratio of minimum yield strength to ultimate tensile strength; for A615 Grade 60 reinforcement

I_g = gross moment of Inertia (in^4)

c = effective slab thickness/2 (in)

M_u = total factored moment, calculated using factored loads described in 18.3.3.1 for Strength I Limit State

Select lowest value of [M_{cr} (or) 1.33 M_u] = M_L

The factored resistance, M_r , or moment capacity, shall be calculated as in 18.3.3.2.1.

Then check that, $M_L \leq M_r$ is satisfied.

$s \le 12.9$ in

Therefore, spacing prov'd. = 7 in < 12.9 in O.K.

Use: #9 at 7" c-c spacing in span 1 (Max. positive reinforcement).

E18-1.7.1.4 Minimum Reinforcement Check

The amount of tensile reinforcement shall be adequate to develop a factored flexural resistance (M_r), or moment capacity, at least equal to the lesser of: **LRFD[5.7.3.3.2]**

M_{cr} (or) $1.33M_u$

$M_{cr} = \gamma_3(\gamma_1 \cdot f_r)S$ where: $S = \frac{I_g}{c}$ therefore, $M_{cr} = 1.1(f_r)\frac{I_g}{c}$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement for A615, Grade 60 reinforcement

$f_r = 0.24\sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r = 0.24\sqrt{4}$ $\boxed{f_r = 0.48}$ ksi

$I_g := \frac{1}{12} \cdot b \cdot d_{slab}{}^3$ $\boxed{I_g = 4913}$ in^4 $c := \frac{d_{slab}}{2}$ $\boxed{c = 8.5}$ in

$M_{cr} = \frac{1.1 f_r \cdot (I_g)}{c} = \frac{1.1 \cdot 0.48 \cdot (4913)}{8.5(12)}$ $\boxed{M_{cr} = 25.43}$ kip-ft

$1.33 \cdot M_u = 138.75$ kip-ft , where M_u was calculated for Strength Design in E18-1.7.1.1 and (M_u = 104.3 kip-ft)

M_{cr} controls because it is less than 1.33 M_u

As shown in E18-1.7.1.1, the reinforcement yields, therefore:

$M_r = 0.90 \cdot A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right)$ $\boxed{M_r = 105}$ kip-ft

Therefore, M_{cr} = 25.43 kip-ft < M_r = 105 kip-ft O.K.

E18-1.7.2 Negative Moment Reinforcement at Piers

Examine at C/L of Pier

E18-1.7.2.1 Design for Strength

Following the procedure in E18-1.7.1.1, using Strength I Limit State:

$$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM}) \leq 0.90\, A_s\, f_s\, (d_s - a/2)$$

The negative live load moment shall be the largest caused by live loads (LL#1, LL#2 or LL#3). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM).

From Table E18.4, the largest live load moment is from (LL#2), therefore at (C/L of Pier):

M_{DC} = -59.2 kip-ft M_{DW} = -4.9 kip-ft M_{LL+IM} = -15.5 + (-39.9) = -55.4 kip-ft

$M_u := 1.25\cdot(-59.2) + 1.50\cdot(-4.9) + 1.75\cdot(-55.4)$ $\boxed{M_u = -178.3}$ kip-ft

$b := 12$ inches (for a one foot design width) and $\boxed{d_s = 25.4}$ in

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_s, are:

$\boxed{R_u = 307.1}$ psi $\boxed{\rho = 0.0054}$ $\boxed{A_s = 1.65}$ $\frac{in^2}{ft}$

Try: #8 at 5 1/2" c-c spacing (A_s = 1.71 in^2/ft) from Table 18.4-4 in 18.4.13

Assume $f_s = f_y$, then the depth of the compressive stress block is: $\boxed{a = 2.51}$ in

Then, $\boxed{c = 2.96}$ in and $\frac{c}{d_s} = 0.12 < 0.6$ therefore, the reinforcement will yield.

The factored resistance is: $\boxed{M_r = 186.6}$ kip-ft

Therefore, M_u = 178.3 kip-ft < M_r = 186.6 kip-ft   O.K.

E18-1.7.2.2 Check for Fatigue

Following the procedure in E18-1.7.1.2, using Fatigue I Limit State:

$1.5\cdot(f_{range}) \leq 24 - 0.33\cdot f_{min}$ (for f_y = 60 ksi)

From Table E18.4, the moments at (C/L Pier) are:

M_{DC} = -59.2 kip-ft M_{DW} = -4.9 kip-ft

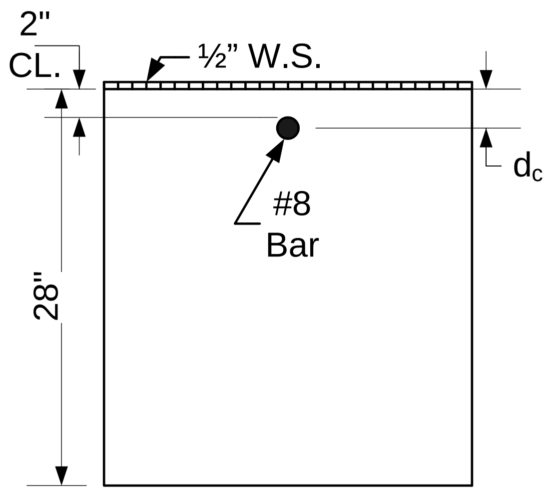

Figure E18.4

Cross Section - (at C/L of Pier)

E18-1.7.2.4 Minimum Reinforcement Check

The amount of tensile reinforcement shall be adequate to develop a factored flexural resistance (M_r), or moment capacity, at least equal to the lesser of: **LRFD [5.7.3.3.2]**

M_{cr} (or) $1.33M_u$

from E18-1.7.1.4, $M_{cr} = 1.1(f_r)\dfrac{I_g}{c}$

Where:

$f_r = 0.24\sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r = 0.24\sqrt{4}$ $\boxed{f_r = 0.48}$ ksi

$I_g := \dfrac{1}{12} \cdot b \cdot D_{haunch}{}^3$ $\boxed{I_g = 21952}$ in^4 $c := \dfrac{D_{haunch}}{2}$ $\boxed{c = 14}$ in

$M_{cr} = \dfrac{1.1 f_r \cdot (I_g)}{c} = \dfrac{1.1 \cdot 0.48 \cdot (21952)}{14(12)}$ $\boxed{M_{cr} = 68.99}$ kip-ft

$1.33 \cdot M_u = 237.1$ kip-ft , where M_u was calculated for Strength Design in E18-1.7.2.1 and (M_u = 178.3 kip-ft)

M_{cr} controls because it is less than 1.33 M_u

By examining E18-1.7.2.1, the reinforcement yields, therefore:

$$M_r = 0.90 \cdot A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right) \qquad \boxed{M_r = 204.1} \text{ kip-ft}$$

Therefore, $M_{cr} = 68.99$ kip-ft < $M_r = 204.1$ kip-ft <u>O.K</u>.

E18-1.7.3 Positive Moment Reinforcement for Span 2

<u>Examine the 0.5 point of span 2</u>

E18-1.7.3.1 Design for Strength

Following the procedure in E18-1.7.1.1, using Strength I Limit State:

$$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM}) \leq 0.90\, A_s\, f_s\, (d_s - a/2)$$

The positive live load moment shall be the largest caused by live loads (LL#1 or LL#2). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM).

From Table E18.4, the largest live load moment is from (LL#1), therefore at (0.5 pt.) of span 2:

$M_{DC} = 19.6$ kip-ft $\quad M_{DW} = 1.6$ kip-ft $\quad M_{LL+IM} = 8.2 + 37.4 = 45.6$ kip-ft

$$M_u := 1.25 \cdot (19.6) + 1.50 \cdot (1.6) + 1.75 \cdot (45.6) \qquad \boxed{M_u = 106.7} \text{ kip-ft}$$

$b := 12$ inches (for a one foot design width) and $\boxed{d_s = 14.9}$ in

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_s, are:

$\boxed{R_u = 534}$ psi $\qquad \boxed{\rho = 0.0097} \qquad \boxed{A_s = 1.73}\ \frac{in^2}{ft}$

Try: #9 at 6" c-c spacing ($A_s = 2.00$ in^2/ft) from Table 18.4-4 in 18.4.13

Assume $f_s = f_y$, then the depth of the compressive stress block is: $\boxed{a = 2.94}$ in

Then, $\boxed{c = 3.46}$ in and $\frac{c}{d_s} = 0.23 < 0.6$ therefore, the reinforcement will yield.

The factored resistance is: $\boxed{M_r = 120.9}$ kip-ft

Therefore, $M_u = 106.7$ kip-ft < $M_r = 120.9$ kip-ft <u>O.K.</u>

$$V_r = \phi_v \cdot V_n = \phi_v \cdot \left(0.063 + \frac{0.126}{\beta_c}\right) \cdot \sqrt{f'_c} \cdot (b_o) \cdot (d_v) \le \phi_v \cdot (0.126) \cdot \sqrt{f'_c} \cdot (b_o) \cdot (d_v)$$

Where:

β_c = ratio of long side to short side of the rectangle through which reaction force is transmitted
$\approx$ 41.71 ft. / 2.5 ft. = 16.7

d_v = effective shear depth = dist. between resultant tensile & compressive forces
$\approx$ 24 in.

b_o = perimeter of the critical section
$\approx$ 1109 in.

Therefore, $V_r := \phi_v \cdot \left(0.063 + \frac{0.126}{\beta_c}\right) \cdot \sqrt{f'_c} \cdot (b_o) \cdot d_v$ $\boxed{V_r = 3380}$ kips

but $\le \phi_v \cdot 0.126 \cdot \sqrt{f'_c} \cdot (b_o) \cdot d_v = 6036$ kips

Therefore, V_u = 1336 kips < V_r = 3380 kips O.K.

Note: Shear check and shear reinforcement design for the pier cap is not shown in this example. Also crack control criteria, minimum reinforcement checks, and shrinkage and temperature reinforcement checks are not shown for the pier cap.

E18-1.16.8 Minimum Reinforcement Check for Transverse Slab Member

Check the negative moment reinforcement (at interior column) for minimum reinforcement criteria.

The amount of tensile reinforcement shall be adequate to develop a factored flexural resistance (M_r), or moment capacity, at least equal to the lesser of: **LRFD [5.7.3.3.2]**

M_{cr} (or) $1.33M_u$

from E18-1.7.1.4, $M_{cr} = 1.1(f_r)\frac{I_g}{c}$

Where:

$f_r = 0.24\sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r = 0.24\sqrt{4}$ $\boxed{f_r = 0.48}$ ksi

h = pier cap depth + D_{haunch} (section depth) $\boxed{h = 58}$ in

b_{cap} = pier cap width $\boxed{b_{cap} = 30}$ in

$I_g := \frac{1}{12} \cdot b_{cap} \cdot h^3$ (gross moment of inertia) $\boxed{I_g = 487780}$ in^4

$c := \frac{h}{2}$ (section depth/2) $\boxed{c = 29}$ in

$$M_{cr} = \frac{1.1 f_r \cdot (I_g)}{c} = \frac{1.1 \cdot 0.48 \cdot (487780)}{29(12)}$$ $\boxed{M_{cr} = 740.1}$ kip-ft

$1.33 \cdot M_u = 605.4$ kip-ft , where M_u was calculated for Strength Design in E18-1.16.6.1 and (M_u = 455.2 kip-ft)

1.33 M_u controls because it is less than M_{cr}

Recalculating requirements for (New moment = $1.33 \cdot M_u$ = 605.4 kip-ft)

$\boxed{b_{neg} = 30}$ in (See E18-1.16.2)

$\boxed{d_{neg} = 54.62}$ in (See E18-1.16.2)

Calculate R_u, coefficient of resistance:

$$R_u = \frac{M_u}{\phi_f \cdot (b_{neg}) \cdot d_{neg}^2} \qquad R_u := \frac{605.4 \cdot (12) \cdot 1000}{0.9(30) \cdot 54.62^2}$$ $\boxed{R_u = 90.2}$ psi

Solve for ρ, reinforcement ratio, using Table 18.4-3 (R_u vs ρ) in 18.4.13;

$\rho := 0.00152$

$A_s = \rho \cdot (b_{neg}) \cdot d_{neg}$ $A_s := 0.00152 \cdot (30) 54.62$ $\boxed{A_s = 2.49}$ in^2

Place this reinforcement in a width, centered over the pier, equal to 1/2 the center to center column spacing or 8 feet, whichever is smaller. Therefore, width equals 6.5 feet.

Therefore, 2.49 in^2/6.5 ft. = 0.38 in^2/ft. Try <u>#5 at 9" c-c spacing</u> for a 6.5 ft. transverse width over the pier. This will provide (A_s = 2.79 in^2) in a 6.5 ft. width.

Calculate the depth of the compressive stress block

Assume $f_s = f_y$ (See 18.3.3.2.1)

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b_{neg}} \qquad a := \frac{2.79 \cdot (60)}{0.85 \cdot (4.0) \cdot 30}$$ $\boxed{a = 1.64}$ in

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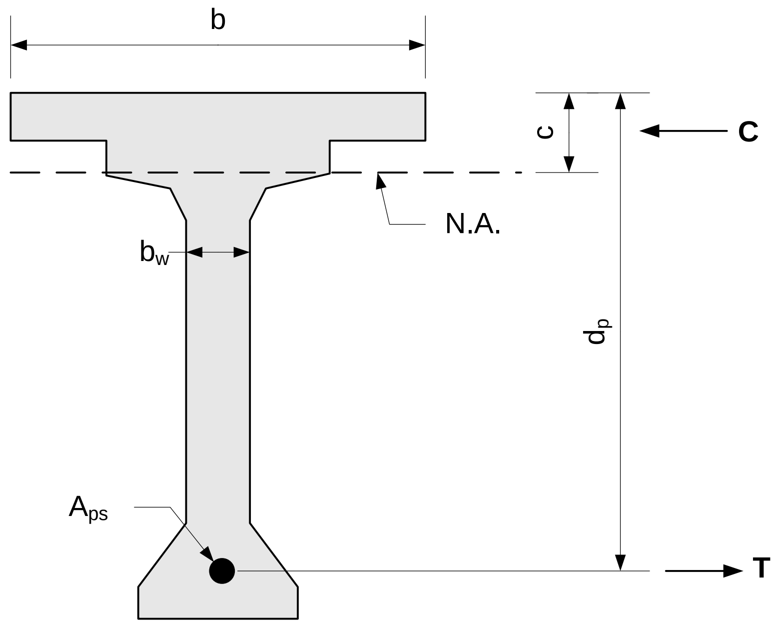


Figure 19.3-3
Depth to Neutral Axis, c

Verify that rectangular section behavior is allowed by checking that the depth of the equivalent stress block, a, is less than or equal to the structural deck thickness. If it is not, then T-section behavior provisions should be followed. If the T-section provisions are used, the compression block will be composed of two different materials with different compressive strengths. In this situation, **LRFD [C5.7.2.2]** recommends using β_1 corresponding to the lower f'_c. The following equation for c shall be used for T-section behavior:

$$c = \frac{A_{ps} f_{pu} - 0.85 f'_c (b - b_w) h_f}{0.85 f'_c \beta_1 b_w + k A_{ps} \dfrac{f_{pu}}{d_p}}$$

Where:

b_w	=	Width of web (in) – use the top flange width if the compression block does not extend below the haunch.
h_f	=	Depth of compression flange (in)

The factored flexural resistance presented in **LRFD [5.7.3.2.2]** is simplified by neglecting the area of mild compression and tension reinforcement. Furthermore, if rectangular section behavior is allowed, then b_w = b, where b_w is the web width as shown in Figure 19.3-3. The equation then reduces to:

$$M_r = \phi A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right)$$

Where:

M_r = Factored flexural resistance (kip-in)

ϕ = Resistance factor

f_{ps} = Average stress in prestressing steel at nominal bending resistance (refer to **LRFD [5.7.3.1.1]**) (ksi)

If the T-section provisions must be used, the factored moment resistance equation is then:

$$M_r = \phi A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + 0.85\phi f'_c (b - b_w) h_f \left(\frac{a}{2} - \frac{h_f}{2}\right)$$

Where:

h_f = Depth of compression flange with width, b (in)

The engineer must then verify that M_r is greater than or equal to M_u.

WisDOT exception to AASHTO:

WisDOT standard prestressed concrete girders and strand patterns are tension-controlled. The ε_t check, as specified in **LRFD [5.7.2.1]**, is not required when the standard girders and strand patterns are used, and $\phi = 1$.

19.3.3.13.2 Minimum Reinforcement

Per **LRFD [5.7.3.3.2]**, the minimum amount of prestressed reinforcement provided shall be adequate to develop an M_r at least equal to the lesser of M_{cr}, or $1.33M_u$.

M_{cr} is the cracking moment, and is given by:

$$M_{cr} = \gamma_3 [S_c (\gamma_1 f_r + \gamma_2 f_{cpe}) - 12M_{dnc} [(S_c/S_{nc}) - 1]]$$

Where:

S_c = Section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (in^3)

f_r = Modulus of rupture (ksi)

f_{cpe} = Compressive stress in concrete due to effective prestress forces only (after losses) at extreme fiber of section where tensile stress is caused by externally applied loads (ksi)

M_{dnc} = Total unfactored dead load moment acting on the basic beam (k-ft)

S_{nc} = Section modulus for the extreme fiber of the basic beam where tensile stress is caused by externally applied loads (in^3)

γ_1 = 1.6 flexural cracking variability factor

γ_2 = 1.1 prestress variability factor

γ_3 = 1.0 for prestressed concrete structures

Per **LRFD [5.4.2.6]**, the modulus of rupture for normal weight concrete is given by:

$$f_r = 0.24\sqrt{f'_c}$$

19.3.3.14 Non-prestressed Reinforcement

Non-prestressed reinforcement consists of bar steel reinforcement used in the conventional manner. It is placed longitudinally along the top of the member to carry any tension which may develop after transfer of prestress. The designer should completely detail all rebar layouts including stirrups.

The amount of reinforcement is that which is sufficient to resist the total tension force in the concrete based on the assumption of an uncracked section.

For draped designs, the control is at the hold-down point of the girder. At the hold-down point, the initial prestress is acting together with the girder dead load stress. This is where tension due to prestress is still maximum and compression due to girder dead load is decreasing.

For non-draped designs, the control is at the end of the member where prestress tension exists but dead load stress does not.

Note that a minimum amount of reinforcement is specified in the Standards. This is intended to help prevent serious damage due to unforeseeable causes like improper handling or storing.

19.3.3.15 Horizontal Shear Reinforcement

The horizontal shear reinforcement resists the Strength I limit state horizontal shear that develops at the interface of the slab and girder in a composite section. The dead load used to calculate the horizontal shear should only consider the DC and DW dead loads that act on the composite section. See 17.2.4 for further information regarding the treatment of dead loads and load combinations.

$$V_u = 1.25DC + 1.50DW + 1.75(LL + IM)$$

$$V_{ni} \geq V_{ui} / \phi$$

Where:

V_u	=	Maximum strength limit state vertical shear (kips)
V_{ui}	=	Strength limit state horizontal shear at the girder/slab interface (kips)
V_{ni}	=	Nominal interface shear resistance (kips)
ϕ	=	0.90 per **LRFD [5.5.4.2.1]**

The shear stress at the interface between the slab and the girder is given by:

$$v_{ui} = \frac{V_u}{b_{vi} d_v}$$

Where:

v_{ui}	=	Factored shear stress at the slab/girder interface (ksi)
b_{vi}	=	Interface width to be considered in shear transfer (in)
d_v	=	Distance between the centroid of the girder tension steel and the mid-thickness of the slab (in)

The factored horizontal interface shear shall then be determined as:

$$V_{ui} = 12 v_{ui} b_{vi}$$

The nominal interface shear resistance shall be taken as:

$$V_{ni} = cA_{cv} + \mu \left[A_{vf} f_y + P_c \right]$$

Where:

A_{cv}	=	Concrete area considered to be engaged in interface shear transfer. This value shall be set equal to $12b_{vi}$ (ksi)
c	=	Cohesion factor specified in **LRFD [5.8.4.3]**. This value shall be taken as 0.28 ksi for WisDOT standard girders with a cast-in-place deck

Where:

f_{pc}	=	Compressive stress in concrete, after all prestress losses, at centroid of cross section resisting externally applied loads or at the web-flange junction when the centroid lies within the flange. (ksi) In a composite member, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the web-flange junction, due to both prestress and moments resisted by the member acting alone.
V_d	=	Shear force at section due to unfactored dead loads (kips)
V_i	=	Factored shear force at section due to externally applied loads occurring simultaneously with M_{max} (kips)
M_{cre}	=	Moment causing flexural cracking at the section due to externally applied loads (k-in)
M_{max}	=	Maximum factored moment at section due to externally applied loads (k-in)

$$V_i = V_u - V_d$$

$$M_{cre} = S_c\left(f_r + f_{cpe} - \frac{12M_{dnc}}{S_{nc}}\right)$$

$$M_{max} = M_u - M_{dnc}$$

Where:

S_c	=	Section modulus for the extreme tensile fiber of the composite section where the stress is caused by externally applied loads (in^3)
S_{nc}	=	Section modulus for the extreme tensile fiber of the noncomposite section where the stress is caused by externally applied loads (in^3)
f_{cpe}	=	Compressive stress in concrete due to effective prestress forces only, after all prestress losses, at the extreme tensile fiber of the section where the stress is caused by externally applied loads (ksi)
M_{dnc}	=	Total unfactored dead load moment acting on the noncomposite section (k-ft)
f_r	=	Modulus of rupture of concrete. Shall be $= 0.20\sqrt{f'_c}$ (ksi)

For a composite section, V_{ci} corresponds to shear at locations of accompanying flexural stress. V_{cw} corresponds to shear at simple supports and points of contraflexure. The critical computation for V_{cw} is at the centroid for composite girders.

Set the vertical component of the draped strands, V_p, equal to 0.0 when calculating V_n, as per **LRFD [5.8.3.3]**. This vertical component helps to reduce the shear on the concrete section. The actual value of V_p should be used when calculating V_{cw}. However, the designer may make the conservative assumption to neglect V_p for all shear resistance calculations.

WisDOT policy item:

Based on past performance, the upper limit for web reinforcement spacing, s_{max}, per **LRFD [5.8.2.7]** will be reduced to 18 inches.

When determining shear reinforcement, spacing requirements as determined by analysis at 1/10th points, for example, should be carried-out to the next 1/10th point. As an illustration, spacing requirements for the 1/10th point should be carried out to very close to the 2/10th point, as the engineer, without a more refined analysis, does not know what the spacing requirements would be at the 0.19 point. For the relatively small price of stirrups, don't shortchange the shear capacity of the prestressed girder.

The web reinforcement spacing shall not exceed the maximum permitted spacing determined as:

- If $\upsilon_u < 0.125f'_c$, then $s_{max} = 0.8d_v \leq 18"$
- If $\upsilon_u \geq 0.125f'_c$, then $s_{max} = 0.4d_v \leq 12"$

Where:

$$\upsilon_u = \frac{V_u - \phi V_p}{\phi b_v d_v}$$ per **LRFD [5.8.2.9]**.

The nominal shear resistance, $V_c + V_s$, is limited by the following:

$$V_c + \frac{A_v f_y d_v \cot\theta}{s} \leq 0.25f'_c b_v d_v$$

Reinforcement in the form of vertical stirrups is required at the extreme ends of the girder. The stirrups are designed to resist 4% of the total prestressing force at transfer at a unit stress of 20 ksi and are placed within h/4 of the girder end, where h is the total girder depth. For a distance of 1.5d from the ends of the beams, reinforcement shall be placed to confine the prestressing steel in the bottom flange. The reinforcement shall be shaped to enclose the strands, shall be a #3 bar or greater and shall be spaced at less than or equal to 6". Note that the reinforcement shown on the Standard Details sheets satisfies these requirements.

E19-1.5.1 Dead Loads

Dead load on non-composite (DC):

exterior:

$$w_{dlxi} := w_g + w_d \cdot \left(\frac{S}{2} + s_{oh}\right) + w_h + 2 \cdot \frac{w_{dx}}{L}$$ $\boxed{w_{dlxi} = 1.706}$ klf

interior:

$$w_{dlii} := w_g + w_d \cdot S + w_h + 2 \cdot \frac{w_{di}}{L}$$ $\boxed{w_{dlii} = 1.834}$ klf

* Dead load on composite (DC):

$$w_p := \frac{2 \cdot w_p}{ng}$$ $\boxed{w_p = 0.129}$ klf

* Wearing Surface (DW):

$$w_{ws} := \frac{w \cdot w_{ws}}{ng}$$ $\boxed{w_{ws} = 0.133}$ klf

* **LRFD [4.6.2.2.1]** states that permanent loads on the deck may be distributed uniformly among the beams. This method is used for the parapet and future wearing surface loads.

E19-1.5.2 Live Loads

For Strength 1 and Service 1 and 3:

HL-93 loading = truck + lane **LRFD [3.6.1.3.1]**

tandem + lane

DLA of 33% applied to truck or tandem, but not to lane per **LRFD [3.6.2.1].**

For Fatigue:

LRFD [5.5.3] states that fatigue of the reinforcement need not be checked for fully prestressed components designed to have extreme fiber tensile stress due to Service III Limit State within the tensile stress limit specified in **LRFD [Table 5.9.4.2.2-1]**.

For fully prestressed components, the compressive stress due to the Fatigue I load combination and one half the sum of effective prestress and permanent loads shall not exceed 0.40 f'c after losses.

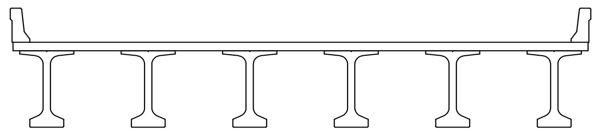
DLA of 15% applied to design truck with a 30 foot axle spacing.

For the Wisconsin Standard Permit Vehicle (Wis-250) Check:

The Wis-250 vehicle is to be checked during the design calculations to make sure it can carry a minimum vehicle weight of 190 kips. See Chapter 45 - Bridge Ratings for calculations.

E19-1.6 Load Distribution to Girders

In accordance with **LRFD [Table 4.6.2.2.1-1]**, this structure is a Type "K" bridge.



Distribution factors are in accordance with **LRFD [Table 4.6.2.2.2b-1]**. For an interior beam, the distribution factors are shown below:

For one Design Lane Loaded:

$$0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\,3}}\right)^{0.1}$$

For Two or More Design Lanes Loaded:

$$0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\,3}}\right)^{0.1}$$

Criteria for using distribution factors - Range of Applicability per **LRFD [Table 4.6.2.2.2b-1]**.

$$\text{DeckSpan} := \left| \begin{array}{ll} \text{"OK"} & \text{if } 3.5 \le S \le 16 \\ \text{"NG"} & \text{otherwise} \end{array} \right.$$

$$\text{DeckThickness} := \left| \begin{array}{ll} \text{"OK"} & \text{if } 4.5 \le t_{se} \le 12 \\ \text{"NG"} & \text{otherwise} \end{array} \right.$$

$$\text{BridgeSpan} := \left| \begin{array}{ll} \text{"OK"} & \text{if } 20 \le L \le 240 \\ \text{"NG"} & \text{otherwise} \end{array} \right.$$

$$\text{NoBeams} := \left| \begin{array}{ll} \text{"OK"} & \text{if } ng \ge 4 \\ \text{"NG"} & \text{otherwise} \end{array} \right.$$

$$\text{LongitStiffness} := \left| \begin{array}{ll} \text{"OK"} & \text{if } 10000 \le K_g \le 7000000 \\ \text{"NG"} & \text{otherwise} \end{array} \right.$$

$$x := \begin{pmatrix} S & \text{DeckSpan} \\ t_{se} & \text{DeckThickness} \\ L & \text{BridgeSpan} \\ ng & \text{NoBeams} \\ K_g & \text{LongitStiffness} \end{pmatrix}$$

$$x = \begin{pmatrix} 7.5 & \text{"OK"} \\ 7.5 & \text{"OK"} \\ 146.0 & \text{"OK"} \\ 6.0 & \text{"OK"} \\ 3600866.5 & \text{"OK"} \end{pmatrix}$$

E19-1.6.1 Distribution Factors for Interior Beams:

One Lane Loaded:

$$g_{i1} := 0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1} \qquad \boxed{g_{i1} = 0.435}$$

Two or More Lanes Loaded:

$$g_{i2} := 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1} \qquad \boxed{g_{i2} = 0.636}$$

$$g_i := \max\left(g_{i1}, g_{i2}\right) \qquad \boxed{g_i = 0.636}$$

Note: The distribution factors above already have a multiple presence factor included that is used for service and strength limit states. For fatigue limit states, the 1.2 multiple presence factor should be divided out.

E19-1.6.2 Distribution Factors for Exterior Beams:

Two or More Lanes Loaded:

Per **LRFD [Table 4.6.2.2.2d-1]** the distribution factor shall be calculated by the following equations:

$$w_{parapet} := \frac{w_b - w}{2}$$ Width of parapet overlapping the deck $\boxed{w_{parapet} = 1.250}$ ft

$$d_e := s_{oh} - w_{parapet}$$ Distance from the exterior web of exterior beam to the interior edge of parapet, ft. $\boxed{d_e = 1.250}$ ft

Note: Conservatively taken as the distance from the center of the exterior girder.

Check range of applicability for de:

$$d_{e\_check} := \begin{cases} \text{"OK"} & \text{if } -1.0 \le d_e \le 5.5 \\ \text{"NG"} & \text{otherwise} \end{cases}$$ $\boxed{d_{e\_check} = \text{"OK"}}$

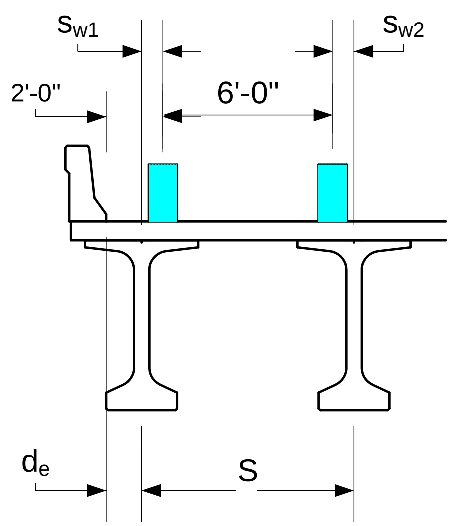
Note: While AASHTO allows the d_e value to be up to 5.5, the deck overhang (from the center of the exterior girder to the edge of the deck) is limited by WisDOT policy as stated in Chapter 17 of the Bridge Manual.

$$e := 0.77 + \frac{d_e}{9.1}$$ $\boxed{e = 0.907}$

$$g_{x2} := e \cdot g_i$$ $\boxed{g_{x2} = 0.577}$

One Lane Loaded:

Per **LRFD [Table 4.6.2.2.2d-1]** the distribution factor shall be calculated by the Lever Rule.



$s_{w1} := d_e - 2$ Distance from center of exterior girder to outside wheel load, ft. $\boxed{s_{w1} = -0.75}$ ft

$s_{w2} := S + s_{w1} - 6$ Distance from wheel load to first interior girder, ft. $\boxed{s_{w2} = 0.75}$ ft

$$R_x := \frac{S + s_{w1} + s_{w2}}{S \cdot 2}$$ $\boxed{R_x = 0.500}$ % of a lane load

Add the single lane multi-presence factor, $m := 1.2$

$$g_{x1} := R_x \cdot 1.2$$ $\boxed{g_{x1} = 0.600}$

The exterior girder distribution factor is the maximum value of the One Lane Loaded case and the Two or More Lanes Loaded case:

$g_x := \max(g_{x1}, g_{x2})$ $\boxed{g_x = 0.600}$

Note: The interior girder has a larger live load distribution factor and a larger dead load than the exterior girder. Therefore, for this example, the interior girder is likely to control.

E19-1.6.3 Distribution Factors for Fatigue:

The distribution factor for fatigue is the single lane distribution factor with the multi-presence factor, $m = 1.200$, removed:

$g_{if} := \frac{g_{i1}}{1.2}$ $\boxed{g_{if} = 0.362}$

E19-1.7 Limit States and Combinations

The limit states, load factors and load combinations shall be applied as required and detailed in chapter 17 of this manual and as indicated below.

E19-1.7.1 Load Factors

From **LRFD [Table 3.4.1-1]**:

	DC	DW	LL
Strength 1	$\gamma st_{DC} := 1.25$	$\gamma st_{DW} := 1.50$	$\gamma st_{LL} := 1.75$
Service 1	$\gamma s1_{DC} := 1.0$	$\gamma s1_{DW} := 1.0$	$\gamma s1_{LL} := 1.0$
Service 3	$\gamma s3_{DC} := 1.0$	$\gamma s3_{DW} := 1.0$	$\gamma s3_{LL} := 0.8$ Check Tension Stress
Fatigue I			$\gamma f_{LL} := 1.50$

Dynamic Load Allowance (IM) is applied to the truck and tandem.

E19-1.7.2 Dead Load Moments

The unfactored dead load moments are listed below (values are in kip-ft):

Unfactored Dead Load Interior Girder Moments (kip-ft)				
	DC	DC	DC	DW
Tenth Point (Along Span)	girder at release	non-composite	composite	composite
0	35	0	0	0
0.1	949	1759	124	128
0.2	1660	3128	220	227
0.3	2168	4105	289	298
0.4	2473	4692	330	341
0.5	2574	4887	344	355

The DC_{nc} values are the component non-composite dead loads and include the weight of the girder, haunch, diaphragms and the deck.

The DC_c values are the component composite dead loads and include the weight of the parapets.

The DW_c values are the composite dead loads from the future wearing surface.

Note that the girder dead load moments at release are calculated based on the girder length. The moments for other loading conditions are calculated based on the span length (center to center of bearing).

E19-1.7.3 Live Load Moments

The unfactored live load load moments (per lane including impact) are listed below (values are in kip-ft). Note that the impact factor is applied only to the truck portion of the HL-93 loads. A separate analysis run will be required if results without impact are desired.

Unfactored Live Load + Impact Moments per Lane (kip-ft)			
Tenth Point	Truck	Tandem	Fatigue
0	0	0	0
0.1	1783	1474	937
0.2	2710	2618	1633
0.3	4100	3431	2118
0.4	4665	3914	2383
0.5	4828	4066	2406

The Wisconsin Standard Permit Vehicle should also be checked. See Chapter 45 - Bridge Rating for further information.

The unfactored live load moments per lane are calculated by applying the appropriate distribution factor to the controlling moment. For the interior girder:

$g_i = 0.636$

$M_{LL} = g_i \cdot 4828$ $\boxed{M_{LL} = 3073}$ kip-ft

$\boxed{g_{if} = 0.362}$

$M_{LLfat} := g_{if} \cdot 2406$ $\boxed{M_{LLfat} = 871}$ kip-ft

E19-1.7.4 Factored Moments

WisDOT's policy is to set all of the load modifiers, η, equal to 1.0. The factored moments for each limit state are calculated by applying the appropriate load factor to the girder moments. For the interior girder:

Strength 1

$$M_{str} := \eta \cdot \left[\gamma st_{DC} \cdot \left(M_{DLnc} + M_{DLc}\right) + \gamma st_{DW} \cdot M_{DWc} + \gamma st_{LL} \cdot M_{LL}\right]$$

$$= 1.0 \cdot \left[1.25 \cdot \left(M_{DLnc} + M_{DLc}\right) + 1.50 \cdot M_{DWc} + 1.75 \cdot M_{LL}\right] \quad \boxed{M_{str} = 12449} \text{ kip-ft}$$

Service 1 (for compression checks)

$$M_{s1} := \eta \cdot \left[\gamma s1_{DC} \cdot \left(M_{DLnc} + M_{DLc}\right) + \gamma s1_{DW} \cdot M_{DWc} + \gamma s1_{LL} \cdot M_{LL}\right]$$

$$= 1.0 \cdot \left[1.0 \cdot \left(M_{DLnc} + M_{DLc}\right) + 1.0 \cdot M_{DWc} + 1.0 \cdot M_{LL}\right] \quad \boxed{M_{s1} = 8659} \text{ kip-ft}$$

Service 3 (for tension checks)

$$M_{s3} := \eta \cdot \left[\gamma s3_{DC} \cdot \left(M_{DLnc} + M_{DLc}\right) + \gamma s3_{DW} \cdot M_{DWc} + \gamma s3_{LL} \cdot M_{LL}\right]$$

$$= 1.0 \cdot \left[1.0 \cdot \left(M_{DLnc} + M_{DLc}\right) + 1.0 \cdot M_{DWc} + 0.8 \cdot M_{LL}\right] \quad \boxed{M_{s3} = 8045} \text{ kip-ft}$$

Service 1 and 3 non-composite DL alone

$$M_{nc} := \eta \cdot \gamma s1_{DC} \cdot M_{DLnc} \quad \boxed{M_{nc} = 4887} \text{ kip-ft}$$

Fatigue 1

$$M_{fat} := \eta \cdot \gamma f_{LL} \cdot M_{LLfat} \quad \boxed{M_{fat} = 1307} \text{ kip-ft}$$

E19-1.8 Composite Girder Section Properties

Calculate the effective flange width in accordance with **LRFD [4.6.2.6]** and section 17.2.11 of the Wisconsin Bridge Manual:

$w_e := S \cdot 12$ $\boxed{w_e = 90.00}$ in

The effective width, w_e, must be adjusted by the modular ratio, n, to convert to the same concrete material (modulus) as the girder.

$w_{eadj} := \frac{w_e}{n}$ $\boxed{w_{eadj} = 58.46}$ in

Calculate the composite girder section properties:

effective slab thickness; $\boxed{t_{se} = 7.50}$ in

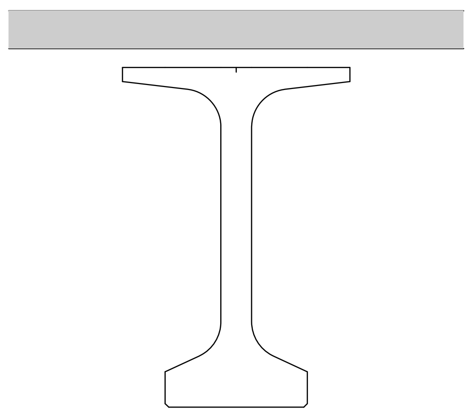
effective slab width; $\boxed{w_{eadj} = 58.46}$ in

haunch thickness; $hau := 2.00$ in

total height; $h_c := ht + hau + t_{se}$

$\boxed{h_c = 81.50}$ in

$\boxed{n = 1.540}$



Note: The area of the concrete haunch is not included in the calculation of the composite section properties.

Component	Ycg	A	AY	AY^2	I	$I+AY^2$
Deck	77.75	438	34088	2650309	2055	2652364
Girder	34.87	915	31906	1112564	656426	1768990
Haunch	73	0	0	0	0	0
Summation		1353	65994			4421354

$\boxed{\Sigma A = 1353}$ in^2

$\boxed{\Sigma AY = 65994}$ in^3

$\boxed{\Sigma IplusAYsq = 4421354}$ in^4

$$y_{cgb} := \frac{-\Sigma AY}{\Sigma A}$$ $\boxed{y_{cgb} = -48.8}$ in

$$y_{cgt} := ht + y_{cgb}$$ $\boxed{y_{cgt} = 23.2}$ in

$$A_{cg} := \Sigma A$$ $\boxed{A_{cg} = 1353}$ in^2

$$I_{cg} := \Sigma IplusAYsq - A_{cg} \cdot y_{cgb}^{\;2}$$ $\boxed{I_{cg} = 1203475}$ in^4

$$S_{cgt} := \frac{I_{cg}}{y_{cgt}}$$ $\boxed{S_{cgt} = 51786}$ in^3

$$S_{cgb} := \frac{I_{cg}}{y_{cgb}}$$ $\boxed{S_{cgb} = -24681}$ in^3

Deck:

$$S_{cgdt} := n \cdot \frac{I_{cg}}{y_{cgt} + hau + t_{se}}$$ $\boxed{S_{cgdt} = 56594}$ in^3

$$S_{cgdb} := n \cdot \frac{I_{cg}}{y_{cgt} + hau}$$ $\boxed{S_{cgdb} = 73411}$ in^3

E19-1.9 Preliminary Design Information:

Controlling Design Criteria

A: At transfer, precasting plant:

T is maximum, little loading
Load = $T_{initial}$ (before losses) + M_g (due to girder weight)

Avoid high initial tension or compression with initial concrete strength.

B: At full service load, final loading (say after 50 years):

T is minimum, load is max
Load = $T_{initial}$ (before losses) + M_g (max service moment)

Avoid cracking and limit concrete stress.

At transfer (Interior Girder):

$M_{iend} := 0$ kip-ft

$M_g := w_g \cdot \frac{L_g^{\,2}}{8}$ $\boxed{M_g = 2574}$ kip-ft

After 50 Years (Interior Girder):

Service 1 Moment $\boxed{M_{s1} = 8659}$ kip-ft

Service 3 Moment $\boxed{M_{s3} = 8045}$ kip-ft

Service 1 Moment Components:

non-composite moment (girder + deck) $\boxed{M_{nc} = 4887}$ kip-ft

composite moment (parapet, FWS and LL)

$M_{1c} := M_{s1} - M_{nc}$ $\boxed{M_{1c} = 3772}$ kip-ft

Service 3 Moment Components:

non-composite moment (girder + deck) $\boxed{M_{nc} = 4887}$ kip-ft

composite moment (parapet, FWS and LL)

$M_{3c} := M_{s3} - M_{nc}$ $\boxed{M_{3c} = 3157}$ kip-ft

At 50 years the prestress has decreased (due to CR, SH, RE):

The approximate method of estimated time dependent losses is used by WisDOT. The lump sum loss estimate, I-girder loss **LRFD [5.9.5.3]**

Where PPR is the partial prestressing ratio, $PPR := 1.0$

$F_{delta} := 33 \cdot \left(1 - 0.15 \cdot \frac{f'_c - 6}{6}\right) + 6 \cdot PPR$ $\boxed{F_{delta} = 37.350}$ ksi

but, for low relaxation strand: $F_{Delta} := F_{delta} - 6$ $\boxed{F_{Delta} = 31.350}$ ksi

Assume an initial strand stress; $f_{tr} := 0.75 \cdot f_{pu}$ $\boxed{f_{tr} = 202.500}$ ksi

Based on experience, assume $\Delta f_{pES\_est} := 18$ ksi loss from elastic shortening. As an alternate initial estimate, **LRFD [C.5.9.5.2.3a]** suggests assuming a 10% ES loss.

$$ES_{loss} := \frac{\Delta f_{pES\_est}}{f_{tr}} \cdot 100 \qquad \boxed{ES_{loss} = 8.889} \ \%$$

$$f_i := f_{tr} - \Delta f_{pES\_est} \qquad \boxed{f_i = 184.500} \ \text{ksi}$$

The total loss is the time dependant losses plus the ES losses:

$$loss := F_{Delta} + \Delta f_{pES\_est} \qquad \boxed{loss = 49.350} \ \text{ksi}$$

$$loss_{\%} := \frac{loss}{f_{tr}} \cdot 100 \qquad \boxed{loss_{\%} = 24.370} \ \%\ \text{(estimated)}$$

If T_o is the initial prestress, then (1-loss)*T_o is the remaining:

$$T = \left(1 - loss_{\%}\right) \cdot T_o$$

$$ratio := 1 - \frac{loss_{\%}}{100} \qquad \boxed{ratio = 0.756}$$

$$T = ratio \cdot T_o$$

E19-1.10 Preliminary Design Steps

The following steps are utilized to design the prestressing strands:

1) Design the amount of prestress to prevent tension at the bottom of the beam under the full load at center span after 50 years.

2) Calculate the prestress losses and check the girder stresses at mid span at the time of transfer.

3) Design the eccentricity of the strands at the girder end to avoid tension or compression over-stress at the time of transfer.

4) If required, design debonding of strands to prevent over-stress at the girder ends.

5) Check resulting stresses at the critical sections of the girder at the time of transfer and after 50 years.

E19-1.10.1 Determine Amount of Prestress

Design the amount of prestress to prevent tension at the bottom of the beam under the full loac (at center span) after 50 years.

Near center span, after 50 years, T = the remaining effective prestress, aim for no tension at the bottom. Use Service I for compression and Service III for tension.

For this example, the interior girder has the controlling moments.

Calculate the stress at the bottom of the beam due to combination of non-composite and composite loading (Service 3 condition):

$$f_b := \frac{M_{nc} \cdot 12}{S_b} + \frac{M_{3c} \cdot 12}{S_{cgb}} \qquad \boxed{f_b = -4.651} \text{ ksi}$$

Stress at bottom due to prestressing (after losses):

$$f_{bp} = \frac{T}{A} \cdot \left(1 + e \cdot \frac{y_b}{r^2}\right) \qquad \text{where } T = \left(1 - loss_{\%}\right) \cdot T_o$$

and $f_{bp} := -f_b$ desired final prestress.

We want this to balance out the tensile stress calculated above from the loading, i.e. an initial compression. Since we are making some assumptions on the actual losses, we are ignoring the allowable tensile stress in the concrete for these calculations.

$$f_{bp} = \frac{\left(1 - loss_{\%}\right) \cdot T_o}{A} \cdot \left(1 + e \cdot \frac{y_b}{r^2}\right) \qquad \text{(after losses)}$$

OR:

$$\frac{f_{bp}}{1 - loss_{\%}} = \frac{T_o}{A} \cdot \left(1 + e \cdot \frac{y_b}{r^2}\right)$$

$$f_{bpi\_1} := \frac{f_{bp}}{1 - \frac{loss_{\%}}{100}}$$ $\boxed{f_{bpi\_1} = 6.149}$ ksi

desired bottom initial prestress (before losses)

If we use the actual allowable tensile stress in the concrete, the desired bottom initial prestress is calculated as follows:

The allowable tension, from **LRFD [5.9.4.2]**, is:

$f_{tall} := 0.19 \cdot \sqrt{f'_c}$ $\boxed{f_{tall} = 0.537}$ ksi

The desired bottom initial prestress (before losses):

$f_{bpi\_2} := f_{bpi\_1} - f_{tall}$ $\boxed{f_{bpi\_2} = 5.612}$ ksi

Determine the stress effects for different strand patterns on the 72W girder:

$A_s = 0.217$ in^2

$f'_s := 270000$ psi

$f_s := 0.75 \cdot f'_s$ $\boxed{f_s = 202500}$ psi

$P := A_s \cdot \frac{f_s}{1000}$ $\boxed{P = 43.943}$ kips

$$f_{bpi} := \frac{P \cdot N}{A_g} \cdot \left(1 + e \cdot \frac{y_b}{r_{sq}}\right)$$ (bottom initial prestress - before losses)

The values of f_{bpi} for various strand patterns is shown in the following table.

72W Stress Effects		
Pi (per strand) = 43.94 kips		
No. Strands	e (in)	bottom stress (ksi)
36	-31.09	4.3411
38	-30.98	4.5726
40	-30.87	4.8030
42	-30.77	5.0333
44	-30.69	5.2648
46	-30.52	5.4858
48	-30.37	5.7075
50	-30.23	5.9290
52	-30.10	6.1504

Solution:
Try $ns := 46$ strands, 0.6 inch diameter.
Initial prestress at bottom $f_{bpi} := 5.4858$ ksi,
Eccentricity, $e_s := -30.52$ inches; actual tension should be less than allowed.

E19-1.10.2 Prestress Loss Calculations

The loss in prestressing force is comprised of the following components:

1) Elastic Shortening (ES), shortening of the beam as soon as prestress is applied.

2) Shrinkage (SH), shortening of the concrete as it hardens, time function.

3) Creep (CR), slow shortening of concrete due to permanent compression stresses in the beam, time function.

4) Relaxation (RE), the tendon slowly accommodates itself to the stretch and the internal stress drops with time

E19-1.10.2.1 Elastic Shortening Loss

at transfer (before ES loss) **LRFD [5.9.5.2]**

$$T_{oi} := ns \cdot f_{tr} \cdot A_s \qquad \boxed{= 46 \cdot 0.75 \cdot 270 \cdot 0.217 = 2021} \quad \text{kips}$$

The ES loss estimated above was: $\Delta f_{pES\_est} = 18.0$ ksi, or $ES_{loss} = 8.889$ %. The resulting force in the strands after ES loss:

$$T_o := \left(1 - \frac{ES_{loss}}{100}\right) \cdot T_{oi} \qquad \boxed{T_o = 1842} \quad \text{kips}$$

If we assume all strands are straight we can calculate the initial elastic shortening loss;

$$f_{cgp} := \frac{T_o}{A_g} + \left(T_o \cdot e_s\right) \cdot \frac{e_s}{I_g} + M_g \cdot 12 \cdot \frac{e_s}{I_g} \qquad \boxed{f_{cgp} = 3.190} \quad \text{ksi}$$

$$\boxed{E_{ct} = 4999} \quad \text{ksi}$$

$$E_p := E_s \qquad \boxed{E_p = 28500} \quad \text{ksi}$$

$$\Delta f_{pES} := \frac{E_p}{E_{ct}} \cdot f_{cgp} \qquad \boxed{\Delta f_{pES} = 18.185} \quad \text{ksi}$$

$f_{tiall} := -\min\left(0.0948 \cdot \sqrt{f'_{ci}}, 0.2\right)$ $\boxed{f_{tiall} = -0.200}$ ksi

bottom:

$$f_{betr} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_b}$$ $\boxed{f_{betr} = 4.994}$ ksi

$\boxed{f_{ciall} = 4.080}$ ksi

high compressive stress

The tension at the top is too high, and the compression at the bottom is also too high!!

Drape some of the strands upward to decrease the top tension and decrease the compression at the bottom.

Find the required position of the steel centroid to avoid tension at the top. Conservatively set the top stress equal to zero and solve for "e":

$$f_{tetr} = \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_t}$$

$$e_{sendt} := \frac{S_t}{T_o} \cdot \left(0 - \frac{T_o}{A_g}\right)$$ $\boxed{e_{sendt} = -19.32}$ inches or higher

Therefore, we need to move the resultant centroid of the strands up:

$move := e_{sendt} - e_s$ $\boxed{move = 11.20}$ inches upward

Find the required position of the steel centroid to avoid high compression at the bottom of the beam. Set the bottom compression equal to the allowable stress and find where the centroid of $ns = 46$ strands needs to be:

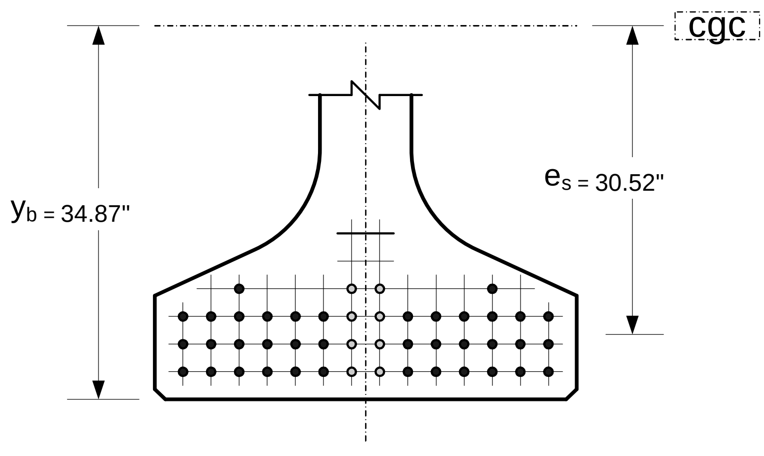
$$f_{betr} = \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_b}$$

Set equal to allowed: $f_{betr} := f_{ciall}$

$$e_{sendb} := \frac{S_b}{T_o} \cdot \left(f_{ciall} - \frac{T_o}{A_g}\right)$$ $\boxed{e_{sendb} = -21.17}$ inches or higher

Top stress condition controls:

$e_{send} := \max\left(e_{sendt}, e_{sendb}\right)$ $\boxed{e_{send} = -19.32}$ inches



LRFD [Table 5.12.3-1] requires 2 inches of cover. However, WisDOT uses 2 inches to the center of the strand, and 2 inch spacing between centers.

The center $ns_d := 8$ strands will be draped at the end of the girder.

Find the center of gravity of the remaining $ns_s = 38$ straight strands from the bottom of the girder:

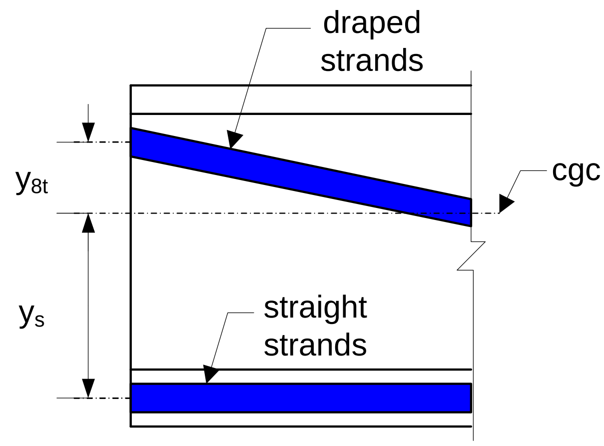
$$Y_s := \frac{12 \cdot 2 + 12 \cdot 4 + 12 \cdot 6 + 2 \cdot 8}{ns_s}$$

$\boxed{Y_s = 4.21}$ inches from the bottom of the girder

OR:

$$y_s := y_b + Y_s$$

$\boxed{y_s = -30.66}$ inches from the center of gravity of the girder (cgc)



y_{8t} is the eccentricity of the draped strands at the end of the beam. We want the eccentricity of all of the strands at the end of the girder to equal, $e_{send} = -19.322$ inches for stress control.

$$e_{send} = \frac{ns_s \cdot y_s + ns_d \cdot y_{8t}}{ns}$$

$$y_{8t} := \frac{ns \cdot e_{send} - ns_s \cdot y_s}{ns_d} \qquad \boxed{y_{8t} = 34.53} \quad \text{inches above the cgc}$$

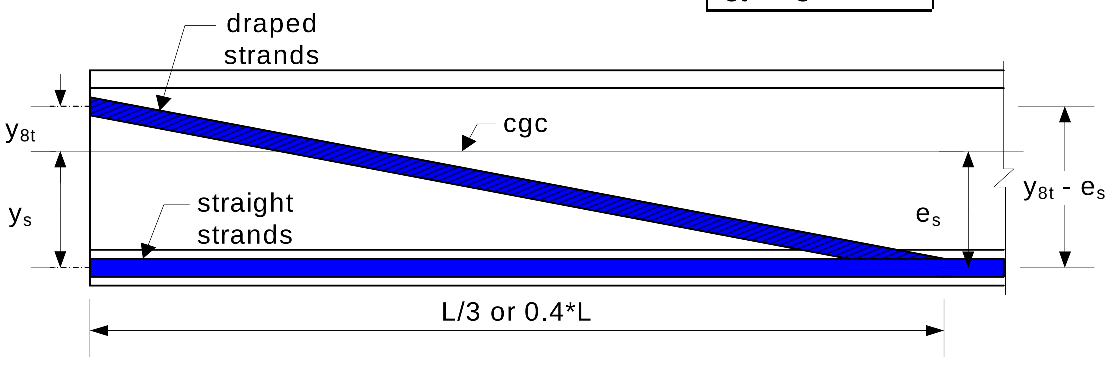
However, $y_t = 37.13$ inches to the top of the beam. If the draped strands are raised $y_{8t} = 34.53$ inches or more above the cgc, the stress will be OK.

Drape the center strands the maximum amount: Maximum drape for $ns_d = 8$ strands:

$$y_{8t} := y_t - 5 \qquad \boxed{y_{8t} = 32.13} \quad \text{in}$$

$$\boxed{e_s = -30.52} \quad \text{in}$$

$$\boxed{y_{8t} - e_s = 62.65} \quad \text{in}$$



Try a drape length of: $\frac{L_g}{3} = 49.00$ feet $\qquad HD := \frac{L_g}{3}$

The eccentricity of the draped strands at the hold down point:

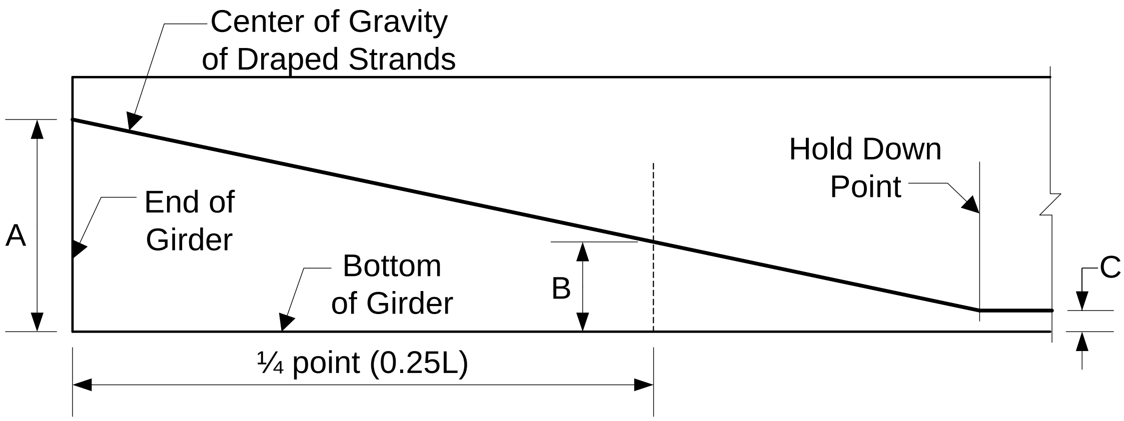
$$e_{8hd} := y_b + 5 \qquad \boxed{e_{8hd} = -29.870} \quad \text{in}$$

Strand slope, $slope := \frac{y_{8t} - e_{8hd}}{(HD \cdot 12)} \cdot 100$ $\boxed{slope = 10.54}$ %

Is the slope of the strands less than 12%? $\boxed{check = "OK"}$

12% is a suggested maximum slope, actual acceptable slope is dependant on the form capacity or on the fabricator.

Calculate the values of A, B_{min}, B_{max} and C to show on the plans:



$A := |y_b| + y_{8t}$ $\boxed{A = 67.00}$ in

$\boxed{C := 5.00}$ in

$B_{min} := \frac{A + 3C}{4}$ $\boxed{B_{min} = 20.50}$ in

$B_{max} := B_{min} + 3$ $\boxed{B_{max} = 23.50}$ in

Check hold down location for B_{max} to make sure it is located between $L_g/3$ and $0.4{*}L_g$:

$slope_{Bmax} := \frac{A - B_{max}}{0.25 \cdot L_g \cdot 12}$ $\boxed{slope_{Bmax} = 0.099}$ ft/ft

$x_{Bmax} := \frac{A - C}{slope_{Bmax}} \cdot \frac{1}{12}$ $\boxed{x_{Bmax} = 52.38}$ ft

$\boxed{L_g \cdot 0.4 = 58.80}$ ft

Is the resulting hold down location less than $0.4{*}L_g$? $\boxed{check = "OK"}$

Check the girder stresses at the end of the transfer length of the strands at release:

Minimum moment on section = girder moment at the plant

The transfer length may be taken as:

$$l_{tr} := 60 \cdot d_b$$ $\boxed{l_{tr} = 36.00}$ in

$$x := \frac{l_{tr}}{12}$$ $\boxed{x = 3.00}$ feet

The eccentricity of the draped strands and the entire strand group at the transfer length is:

$$y_{8tt} := y_{8t} - \frac{slope}{100} \cdot x \cdot 12$$ $\boxed{y_{8tt} = 28.334}$ in

$$e_{st} := \frac{ns_s \cdot y_s + 8 \cdot y_{8tt}}{ns}$$ $\boxed{e_{st} = -20.400}$ in

The moment at the end of the transfer length due to the girder dead load:

$$M_{gt} := \frac{w_g}{2} \cdot \left(L_g \cdot x - x^2\right)$$ $\boxed{M_{gt} = 206}$ kip-ft

The girder stresses at the end of the transfer length:

$$f_{tt} := \frac{T_o}{A_g} + \frac{T_o \cdot e_{st}}{S_t} + \frac{M_{gt} \cdot 12}{S_t}$$ $\boxed{f_{tt} = 0.028}$ ksi

$\boxed{f_{tiall} = -0.200}$ ksi

Is f_{tt} less than f_{tiall}? $\boxed{check = \text{"OK"}}$

$$f_{bt} := \frac{T_o}{A_g} + \frac{T_o \cdot e_{st}}{S_b} + \frac{M_{gt} \cdot 12}{S_b}$$ $\boxed{f_{bt} = 3.873}$ ksi

$\boxed{f_{ciall} = 4.080}$ ksi

Is f_{bt} less than f_{ciall}? $\boxed{check = \text{"OK"}}$

E19-1.10.4 Stress Checks at Critical Sections

Critical Sections	Critical Conditions		
	At Transfer	Final	Fatigue
Girder Ends	X		
Midspan	X	X	X
Hold Down Points	X	X	X

Data:

$T_o = 1840$ kips $\quad$ $T = 1602$ kips

$M_{nc} = 4887$ kip-ft $\quad$ $M_{s3} = 8045$ kip-ft

$M_{s1} = 8659$ kip-ft $\quad$ $M_g = 2574$ kip-ft

Need moments at hold down points: $\frac{L_g}{3} = 49.00$ feet, from the end of the girder.

girder: $M_{ghd} = 2288$ kip-ft

non-composite: $M_{nchd} = 4337$ kip-ft

Service I composite: $M_{1chd} = 3371$ kip-ft

Service III composite: $M_{3chd} = 2821$ kip-ft

Note: The release girder moments shown above at the hold down location are calculated basec on the total girder length.

Check the girder at the end of the beam (at the transfer length):

$e_{st} = -20.40$ inches $\quad$ $f_{tiall} = -0.200$ ksi $\quad$ $f_{ciall} = 4.080$ ksi

At transfer, $M_{gt} = 206$ kip-ft

Top of girder (Service 3):

$$f_{tei} := \frac{T_o}{A_g} + \frac{T_o \cdot e_{st}}{S_t} + \frac{M_{gt} \cdot 12}{S_t}$$

$f_{tei} = 0.028$ ksi

Is f_{tei} greater than f_{tiall}? $\quad$ check = "OK"

Bottom of girder (Service 1):

$$f_{bei} := \frac{T_o}{A_g} + \frac{T_o \cdot e_{st}}{S_b} + \frac{M_{gt} \cdot 12}{S_b}$$

$f_{bei} = 3.873$ ksi

Is f_{bei} less than f_{ciall}? $\quad$ check = "OK"

Check at the girder and deck at midspan:

$\boxed{e_s = -30.52}$ inches

Initial condition at transfer: $\boxed{f_{tiall} = -0.200}$ ksi $\boxed{f_{ciall} = 4.080}$ ksi

Top of girder stress (Service 3):

$$f_{ti} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_t} + \frac{M_g \cdot 12}{S_t}$$ $\boxed{f_{ti} = 0.582}$ ksi

Is f_{ti} greater than f_{tiall}? $\boxed{check = "OK"}$

Bottom of girder stress (Service 1):

w

$$f_{bi} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_b} + \frac{M_g \cdot 12}{S_b}$$ $\boxed{f_{bi} = 3.353}$ ksi

Is f_{bi} less than f_{ciall}? $\boxed{check = "OK"}$

Final condition:

Allowable Stresses,**LRFD [5.9.4.2]**:

There are two compressive stress limits: (Service 1) **LRFD [5.9.4.2.1]**

$f_{call1} := 0.45 \cdot f'_c$ PS + DL $\boxed{f_{call1} = 3.600}$ ksi

$f_{call2} := 0.60 \cdot f'_c$ LL + PS + DL $\boxed{f_{call2} = 4.800}$ ksi

(Service 3) **LRFD [5.9.4.2.2]** (Moderate Corrosion Condition)

tension: $f_{tall} := -0.19 \cdot \sqrt{f'_c}$ $\boxed{f_{tall} = -0.537}$ ksi

Allowable Stresses (Fatigue),**LRFD [5.5.3]**:

Fatige compressive stress limit:

$f_{call\_fat} := 0.40 \cdot f'_c$ LLfat + 1/2(PS + DL) $\boxed{f_{call\_fat} = 3.200}$ ksi

Top of girder stress (Service 1):

$$f_{t1} := \frac{T}{A_g} + \frac{T \cdot e_s}{S_t} + \frac{M_{DLnc} \cdot 12}{S_t} + \frac{(M_{DLc} + M_{DWc}) \cdot 12}{S_{cgt}}$$

$\boxed{f_{t1} = 2.465}$ ksi

$$f_{t2} := \frac{T}{A_g} + \frac{T \cdot e_s}{S_t} + \frac{M_{DLnc} \cdot 12}{S_t} + \frac{(M_{DLc} + M_{DWc} + M_{LL}) \cdot 12}{S_{cgt}}$$

$\boxed{f_{t2} = 3.177}$ ksi

Is f_t less than f_{call}?

$\boxed{check1 = \text{"OK"}}$

$\boxed{check2 = \text{"OK"}}$

Top of girder stress (Fatigue 1):

$$f_{tfat} := \frac{1}{2}\left(\frac{T}{A_g} + \frac{T \cdot e_s}{S_t} + \frac{M_{DLnc} \cdot 12}{S_t}\right) + \frac{\left[\frac{1}{2}(M_{DLc} + M_{DWc}) + M_{LLfat}\right] \cdot 12}{S_{cgt}}$$

$\boxed{f_{tfat} = 1.434}$ ksi

Is f_{tfat} less than f_{call_fat}?

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

Bottom of girder stress (Service 3):

$$f_b := \frac{T}{A_g} + \frac{T \cdot e_s}{S_b} + \frac{M_{nc} \cdot 12}{S_b} + \frac{(M_{s3} - M_{nc}) \cdot 12}{S_{cgb}}$$

$\boxed{f_b = -0.302}$ ksi

Is f_{tb} greater than f_{tall}?

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

Top of deck stress (Service 1):

$$f_{dall} := 0.40 \cdot f'_{cd}$$

$\boxed{f_{dall} = 1.600}$ ksi

where:

$A_{ps} := ns \cdot A_s$ $\boxed{A_{ps} = 9.98}$ in^2

$b := w_e$ $\boxed{b = 90.00}$ in

LRFD [5.7.2.2]

$\beta_1 := \max\left[0.85 - \left(f'_{cd} - 4\right) \cdot 0.05, 0.65\right]$ $\boxed{\beta_1 = 0.850}$

$d_p := y_t + hau + t_{se} - e_s$ $\boxed{d_p = 77.15}$ in

$$c := \frac{A_{ps} \cdot f_{pu}}{0.85 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c = 9.99}$ in

$a := \beta_1 \cdot c$ $\boxed{a = 8.49}$ in

The calculated value of "a" is greater than the deck thickness. Therefore, the rectangular assumption is incorrect and the compression block extends into the haunch. Calculate the neutral axis location and capacity for a flanged section:

$h_f := t_{se}$ depth of compression flange $\boxed{h_f = 7.500}$ in

$w_{tf} = 48.00$ width of top flange, inches

$$c := \frac{A_{ps} \cdot f_{pu} - 0.85 \cdot f'_{cd} \cdot \left(b - w_{tf}\right) \cdot h_f}{0.85 \cdot f'_{cd} \cdot \beta_1 \cdot w_{tf} + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c = 10.937}$ in

$a := \beta_1 \cdot c$ $\boxed{a = 9.30}$ in

This is within the depth of the haunch (9.5 inches). Therfore our assumption is OK.

Now calculate the effective tendon stress at ultimate:

$$f_{ps} := f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right)$$ $\boxed{f_{ps} = 259.283}$ ksi

$T_u := f_{ps} \cdot A_{ps}$ $\boxed{T_u = 2588}$ kips

Calculate the nominal moment capacity of the composite section in accordance with **LRFD [5.7.3.2]**:

$$M_n := \left[A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) + 0.85 \cdot f'_{cd} \cdot \left(b - w_{tf}\right) \cdot h_f \cdot \left(\frac{a}{2} - \frac{h_f}{2}\right)\right] \cdot \frac{1}{12}$$

$\boxed{M_n = 15717}$ kip-ft

For prestressed concrete, $\phi_f := 1.00$, **LRFD [5.5.4.2.1]**. Therefore the usable capacity is:

$M_r := \phi_f \cdot M_n$ $\boxed{M_r = 15717}$ kip-ft

The required capacity:

Interior Girder Moment $\boxed{M_{str} = 12449}$ kip-ft

Exterior Girder Moment $\boxed{M_{strx} = 11183}$ kip-ft

Check the section for minimum reinforcement in accordance with **LRFD [5.7.3.3.2]** for the interior girder:

$\boxed{1.33 \cdot M_{str} = 16558}$ kip-ft

$f_r := 0.24 \cdot \sqrt{f'_c}$ **LRFD [5.4.2.6]** $\boxed{f_r = 0.679}$ ksi

$$f_{cpe} := \frac{T}{A_g} + \frac{T \cdot e_s}{S_b}$$ $\boxed{f_{cpe} = 4.348}$ ksi

$M_{dnc} := M_{nc}$ $\boxed{M_{dnc} = 4887}$ kip-ft

$S_c := -S_{cgb}$ $\boxed{S_c = 24681}$ in^3

$S_{nc} := -S_b$ $\boxed{S_{nc} = 18825}$ in^3

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_2 := 1.1$ prestress variability factor

$\gamma_3 := 1.0$ for prestressed concrete structures

$$M_{cr} := \gamma_3 \cdot \left[S_c \cdot \left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe} \right) \cdot \frac{1}{12} - M_{dnc} \cdot \left(\frac{S_c}{S_{nc}} - 1 \right) \right]$$ $\boxed{M_{cr} = 10551}$ kip-ft

Is M_r greater than the lesser value of M_{cr} and $1.33*M_{str}$? $\boxed{check = "OK"}$

The moment capacity looks good, with some over strength for the interior girder. However, we must check the capacity of the exterior girder since the available flange width is less.

Check the exterior girder capacity:

The effective flange width for exterior girder is calculated in accordance with **LRFD [4.6.2.6]** as one half the effective width of the adjacent interior girder plus the overhang width :

$$w_{ex\_oh} := s_{oh} \cdot 12 \qquad \boxed{w_{ex\_oh} = 30.0} \text{ in}$$

$$w_{ex} := \frac{w_e}{2} + w_{ex\_oh} \qquad \boxed{w_{ex} = 75.00} \text{ in}$$

$b_x := w_{ex}$ effective deck width of the compression flange.

Calculate the neutral axis location for a flanged section:

$$c_x := \frac{A_{ps} \cdot f_{pu} - 0.85 \cdot f'_{cd} \cdot (b_x - w_{tf}) \cdot h_f}{0.85 \cdot f'_{cd} \cdot \beta_1 \cdot w_{tf} + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}} \qquad \boxed{c_x = 13.51} \text{ in}$$

$$a_x := \beta_1 \cdot c_x \qquad \boxed{a_x = 11.49} \text{ in}$$

Now calculate the effective tendon stress at ultimate:

$$f_{ps\_x} := f_{pu} \cdot \left(1 - k \cdot \frac{c_x}{d_p}\right) \qquad \boxed{f_{ps\_x} = 256.759} \text{ ksi}$$

The nominal moment capacity of the composite section (exterior girder) ignoring the increased strength of the concrete in the girder flange:

$$M_{n\_x} := \left[A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a_x}{2}\right) + 0.85 \cdot f'_{cd} \cdot (b_x - w_{tf}) \cdot h_f \cdot \left(\frac{a_x}{2} - \frac{h_f}{2}\right)\right] \cdot \frac{1}{12}$$

$$\boxed{M_{n\_x} = 15515} \text{ kip-ft}$$

$$M_{r\_x} := \phi_f \cdot M_{n\_x} \qquad \boxed{M_{r\_x} = 15515} \text{ kip-ft}$$

$\boxed{1.33M_{strx} = 14874}$ kip-ft

Is M_{r_x} greater than 1.33*M_{strx}? $\boxed{check = "OK"}$

Since M_{r_x} is greater than 1.33*M_{strx}, the check for M_{cr} does not need to be completed.

E19-1.13 Shear Analysis

A separate analysis must be conducted to estimate the total shear force in each girder for shear design purposes.

Calculate the shear distribution to the girders, **LRFD [Table 4.6.2.2.3a-1]**:

Interior Beams:

One lane loaded:

$$g_{vi1} := 0.36 + \frac{S}{25} \qquad \boxed{g_{vi1} = 0.660}$$

Two or more lanes loaded:

$$g_{vi2} := 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2 \qquad \boxed{g_{vi2} = 0.779}$$

$$g_{vi} := \max\left(g_{vi1}, g_{vi2}\right) \qquad \boxed{g_{vi} = 0.779}$$

Note:The distribution factors above include the multiple lane factor. *The skew correction factor, as now required by a WisDOT policy item for all girders, is omitted. This example is not yet revised.*

Exterior Beams:

Two or more lanes loaded:

The distance from the centerline of the exterior beam to the inside edge of the parapet, $d_e = 1.25$ feet.

$$e_v := 0.6 + \frac{d_e}{10} \qquad \boxed{e_v = 0.725}$$

$$g_{vx2} := e_v \cdot g_{vi} \qquad \boxed{g_{vx2} = 0.565}$$

With a single lane loaded, we use the lever rule (same as before). Note that the multiple presence factor has already been applied to g_{x2}..

$$g_{vx1} := g_{x1} \quad = e \cdot g_i \qquad \boxed{g_{vx1} = 0.600}$$

$g_{vx} := \max(g_{vx1}, g_{vx2})$ $\boxed{g_{vx} = 0.600}$

Apply the shear magnification factor in accordance with **LRFD [4.6.2.2.3c]**.

$$skew_{correction} := 1.0 + 0.2 \cdot \left(\frac{12L \cdot t_s^{\,3}}{K_g}\right)^{0.3} \cdot \tan\left(skew \cdot \frac{\pi}{180}\right)$$

$\boxed{L = 146.00}$

$\boxed{t_s = 8.00}$

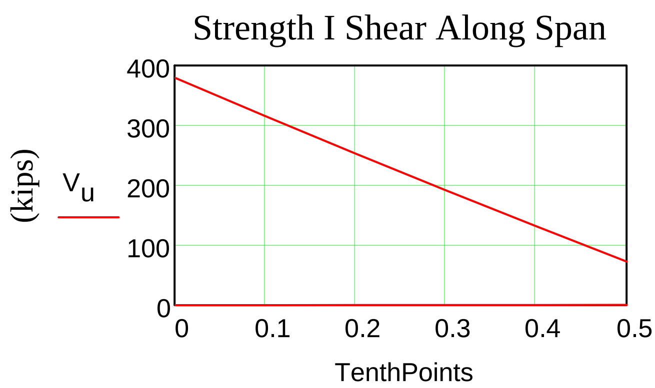
$\boxed{K_g = 3600866}$

$\boxed{skew = 20.000}$ $\boxed{skew_{correction} = 1.048}$

$g_{vx} := g_{vx} \cdot skew_{correction}$ $\boxed{g_{vx} = 0.629}$

The interior girder will control. It has a larger distribution factor and a larger dead load.

Conduct a bridge analysis as before with similar load cases for the maximum girder shear forces. We are interested in the Strength 1 condition now for shear design.



$\boxed{Vu_{0.0} = 379.7}$ kips

$\boxed{Vu_{0.5} = 72.9}$ kips

Simplified Procedure for Prestressed and Nonprestressed Sections, **LRFD [5.8.3.4.3]**

$b_v := t_w$ $\boxed{b_v = 6.50}$ in

The critical section for shear is taken at a distance of d_v from the face of the support, **LRFD [5.8.3.2]**.

d_v = effective shear depth taken as the distance between the resultants of the tensile and compressive forces due to flexure. It need not be taken less than the greater of 0.9*d_e or 0.72h (inches). **LRFD [5.8.2.9]**

The first estimate of d_v is calculated as follows:

$$d_v := -e_s + y_t + hau + t_{se} - \frac{a}{2} \qquad \boxed{d_v = 72.50} \text{ in}$$

However, since there are draped strands for a distance of $HD = 49.00$ feet from the end of the girder, a revised value of e_s should be calculated based on the estimated location of the critical section. Since the draped strands will raise the center of gravity of the strand group near the girder end, try a smaller value of "d_v" and recalculate "e_s" and "a".

Try $d_v := 65$ inches.

For the standard bearing pad of width, $w_{brg} := 8$ inches, the distance from the end of the girder to the critical section:

$$L_{crit} := \left(\frac{w_{brg}}{2} + d_v\right) \cdot \frac{1}{12} + 0.5 \qquad \boxed{L_{crit} = 6.25} \text{ ft}$$

Calculate the eccentricity of the strand group at the critical section.

$$y_{8t\_crit} := y_{8t} - \frac{slope}{100} \cdot L_{crit} \cdot 12 \qquad \boxed{y_{8t\_crit} = 24.22} \text{ in}$$

$$e_{s\_crit} := \frac{ns_s \cdot y_s + ns_d \cdot y_{8t\_crit}}{ns_s + ns_d} \qquad \boxed{e_{s\_crit} = -21.11} \text{ in}$$

Calculation of compression stress block based on revised eccentricity:

$$d_{p\_crit} := y_t + hau + t_{se} - e_{s\_crit} \qquad \boxed{d_{p\_crit} = 67.74} \text{ in}$$

$$A_{ps\_crit} := (ns_d + ns_s) \cdot A_s \qquad \boxed{A_{ps\_crit} = 9.98} \text{ in}^2$$

Also, the value of f_{pu}, should be revised if the critical section is located less than the development length from the end of the beam. The development length for a prestressing strand is calculated in accordance with **LRFD [5.11.4.2]**:

$K := 1.6$ for prestressed members with a depth greater than 24 inches

$f_r := -0.20 \cdot \sqrt{f'_c}$ **LRFD [5.4.2.6]** $\boxed{f_r = -0.566}$ ksi

$\boxed{T = 1602}$ kips

$f_{cpe} := \frac{T}{A_g} + \frac{T \cdot e_{s\_crit}}{S_b}$ $\boxed{f_{cpe} = 3.548}$ ksi

$\boxed{M_{dnc} = 740}$ kip-ft

$\boxed{M_{max} = 10048}$ kip-in

$S_c := S_{cgb}$ $\boxed{S_c = -24681}$ in^3

$S_{nc} := S_b$ $\boxed{S_{nc} = -18825}$ in^3

$M_{cre} := S_c \cdot \left(f_r - f_{cpe} - \frac{12M_{dnc}}{S_{nc}} \right)$ $\boxed{M_{cre} = 89892}$ kip-in

$V_{ci1} := 0.06 \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$ $\boxed{V_{ci1} = 71.7}$ kips

$V_{ci2} := 0.02 \cdot \sqrt{f'_c} \cdot b_v \cdot d_v + V_d + \frac{V_i \cdot M_{cre}}{M_{max}}$ $\boxed{V_{ci2} = 1384.0}$ kips

$V_{ci} := \max(V_{ci1}, V_{ci2})$ $\boxed{V_{ci} = 1384.0}$ kips

$f_t := \frac{T}{A_g} + \frac{T \cdot e_{s\_crit}}{S_t} + \frac{M_{dnc} \cdot 12}{S_t}$ $\boxed{f_t = 0.340}$ ksi

$f_b := \frac{T}{A_g} + \frac{T \cdot e_{s\_crit}}{S_b} + \frac{M_{dnc} \cdot 12}{S_b}$ $\boxed{f_b = 3.076}$ ksi

$\boxed{y_{cgb} = -48.76}$ in

$\boxed{ht = 72.00}$ in

$f_{pc} := f_b - y_{cgb} \cdot \frac{f_t - f_b}{ht}$ $\boxed{f_{pc} = 1.223}$ ksi

$V_{p\_cw} := ns_d \cdot A_s \cdot f_{pe} \cdot \frac{slope}{100}$ $\boxed{V_{p\_cw} = 29.4}$ kips

$V_{cw} := \left(0.06 \cdot \sqrt{f'_c} + 0.30 \cdot f_{pc}\right) \cdot b_v \cdot d_v + V_{p\_cw}$ $\boxed{V_{cw} = 256.1}$ kips

$V_c := \min\left(V_{ci}, V_{cw}\right)$ $\boxed{V_c = 256.1}$ kips

Calculate the required shear resistance:

$\phi_v := 0.9$ **LRFD [5.5.4.2]**

$V_{u\_crit} = \gamma st_{DC} \cdot \left(V_{DCnc} + V_{DCc}\right) + \gamma st_{DW} \cdot V_{DWc} + \gamma st_{LL} \cdot Vu_{LL}$ where,

$V_{DCnc} = 123.357$ kips $V_{DCc} = 8.675$ kips $V_{DWc} = 8.967$ kips $Vu_{LL} = 100.502$ kips

$V_{u\_crit} = 354.368$ kips $V_n := \frac{V_{u\_crit}}{\phi_v}$ $\boxed{V_n = 393.7}$ kips

Transverse Reinforcing Design at Critical Section:

The required steel capacity:

$V_s := V_n - V_c - V_p$ $\boxed{V_s = 137.7}$ kips

$A_v := 0.40$ in^2 for #4 rebar

$f_y := 60$ ksi

$\boxed{d_v = 65.00}$ in

$$\cot\theta := \begin{cases} 1 & \text{if } V_{ci} < V_{cw} \\ \min\left(1.0 + 3 \cdot \frac{f_{pc}}{\sqrt{f'_c}}, 1.8\right) & \text{otherwise} \end{cases}$$

$\boxed{\cot\theta = 1.800}$

$V_s = A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{s}$ **LRFD Eq 5.8.3.3-4** reduced per **C5.8.3.3-1** when α = 90 degrees.

$s := A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{V_s}$ $\boxed{s = 20.399}$ in

Check Maximum Spacing, **LRFD [5.8.2.7]**:

$$v_u := \frac{V_{u\_crit}}{\phi_v \cdot b_v \cdot d_v}$$ $\boxed{v_u = 0.932}$ ksi

Max. stirrup spacing per WisDOT policy item is 18" $\boxed{0.125 \cdot f'_c = 1.000}$ ksi

$$s_{max1} := \begin{cases} \min(0.8 \cdot d_v, 18) & \text{if } v_u < 0.125 \cdot f'_c \\ \min(0.4 \cdot d_v, 12) & \text{if } v_u \geq 0.125 \cdot f'_c \end{cases}$$ $\boxed{s_{max1} = 18.00}$ in

Check Minimum Reinforcing, **LRFD [5.8.2.5]**:

$$s_{max2} := \frac{A_v \cdot f_y}{0.0316 \cdot \sqrt{f'_c} \cdot b_v}$$ $\boxed{s_{max2} = 41.31}$ in

$$s_{max} := \min(s_{max1}, s_{max2})$$ $\boxed{s_{max} = 18.00}$ in

Therefore use a maximum spacing of $s := 18$ inches.

$$V_s := A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{s}$$ $\boxed{V_s = 156}$ kips

Check V_n requirements:

$$Vn_1 := V_c + V_s + V_p$$ $\boxed{Vn_1 = 412}$ kips

$$Vn_2 := 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p$$ $\boxed{Vn_2 = 845}$ kips

$$V_n := \min(Vn_1, Vn_2)$$ $\boxed{V_n = 412}$ kips

$$V_r := \phi_v \cdot V_n$$ $\boxed{V_r = 370.88}$ kips

$\boxed{V_{u\_crit} = 354.37}$ kips

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

Web reinforcing is required in accordance with **LRFD [5.8.2.4]** whenever:

$$V_u \geq 0.5 \cdot \phi_v \cdot (V_c + V_p)$$ (all values shown are in kips)

At critical section from end of girder: $\boxed{V_{u\_crit} = 354}$ $\boxed{0.5 \cdot \phi_v \cdot (V_c + V_p) = 115}$

From calculations similar to those shown above,

At hold down point: $\boxed{V_{u\_hd} = 172}$ $\boxed{0.5 \cdot \phi_v \cdot (V_{c\_hd} + V_p) = 64}$

At mid-span: $\boxed{V_{u\_mid} = 73}$ $\boxed{0.5 \cdot \phi_v \cdot (V_{c\_mid} + V_p) = 36}$

Therefore, use web reinforcing over the entire beam.

Resulting Shear Design:

Use #4 U shaped stirrups at 18-inch spacing between the typical end sections. Unless a large savings in rebar can be realized, use a single stirrup spacing between the standard end sections.

E19-1.14 Longitudinal Tension Flange Capacity:

The total capacity of the tension reinforcing must meet the requirements of **LRFD [5.8.3.5].** The capacity is checked at the critical section for shear:

$$T_{ps} := \frac{M_{max}}{d_v \cdot \phi_f} + \left(\frac{V_{u\_crit}}{\phi_v} - 0.5 \cdot V_s - V_{p\_cw} \right) \cdot \cot\theta \qquad \boxed{T_{ps} = 670} \text{ kips}$$

actual capacity of the straight strands:

$\boxed{ns_S \cdot A_s \cdot f_{pu\_crit} = 1612}$ kips

Is the capacity of the straight strands greater than T_{ps}? $\boxed{check = \text{"OK"}}$

Check the tension Capacity at the edge of the bearing:

The strand is anchored $l_{px} := 10$ inches. The transfer and development lengths for a prestressing strand are calculated in accordance with **LRFD [5.11.4.2]**:

$\boxed{l_{tr} = 36.00}$ in

$\boxed{l_d = 146.2}$ in

Since l_{px} is less than the transfer length, the design stress in the prestressing strand is calculated as follows:

The assumed crack plane crosses the centroid of the straight strands at

$l_{px'} := l_{px} + Y_s \cdot \cot\theta$ $\boxed{Y_s = 4.21}$ in $\boxed{l_{px'} = 17.58}$ in

$$f_{pb} := \frac{f_{pe} \cdot l_{px'}}{60 \cdot d_b}$$ $\boxed{f_{pb} = 78.37}$ ksi

Tendon capacity of the straight strands: $\boxed{ns_s \cdot A_s \cdot f_{pb} = 646}$ kips

The values of V_u, V_s, V_p and θ may be taken at the location of the critical section.

Over the length d_v, the average spacing of the stirrups is:

$$s_{ave} := \frac{6 \cdot 4.25 + 6 \cdot 5.5}{12}$$ $\boxed{s_{ave} = 4.88}$ in

$$V_s := A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{s_{ave}}$$ $\boxed{V_s = 576}$ kips

The vertical component of the draped strands is: $\boxed{V_{p\_cw} = 29}$ kips

The factored shear force at the critical section is: $\boxed{V_{u\_crit} = 354}$ kips

Minimum capacity required at the front of the bearing:

$$T_{breqd} := \left(\frac{V_{u\_crit}}{\phi_v} - 0.5 \cdot V_s - V_{p\_cw}\right) \cdot \cot\theta$$ $\boxed{T_{breqd} = 137}$ kips

Is the capacity of the straight strands greater than T_{breqd}? $\boxed{check = "OK"}$

E19-1.15 Composite Action - Design for Interface Shear Transfer

The total shear to be transferred to the flange between the end of the beam and mid-span is equal to the compression force in the compression block of the flange and haunch in strength condition. For slab on girder bridges, the shear interface force is calculated in accordance with **LRFD [5.8.4.2]**.

$b_{vi} := 18$ in width of top flange available to bond to the deck

$\boxed{d_v = 65.00}$ in

$$v_{ui} := \frac{V_{u\_crit}}{b_{vi} \cdot d_v}$$ $\boxed{v_{ui} = 0.303}$ ksi

$$V_{ui} := v_{ui} \cdot 12 \cdot b_{vi}$$ $\boxed{V_{ui} = 65.4}$ kips/ft

$V_n = c \cdot A_{cv} + \mu \cdot \left(A_{vf} \cdot f_y + P_c\right)$ **LRFD [5.8.4.1]**

The nominal shear resistance, V_n, used in design shall not be greater than the lesser of:

$V_{n1} = K_1 \cdot f'_{cd} \cdot A_{cv}$ or $V_{n2} = K_2 \cdot A_{cv}$

$c := 0.28$ ksi

$\mu := 1.0$

$K_1 := 0.3$

$K_2 := 1.8$

$A_{cv} := b_{vi} \cdot 12$ Area of concrete considered to be engaged in interface shear transfer. $\boxed{A_{cv} = 216}$ in^2/ft

For an exterior girder, P_c is the weight of the deck, haunch, parapet and FWS.

$$P_{cd} := \frac{w_c \cdot \frac{t_s}{12}}{2 \cdot S} \cdot \left(S + s_{oh}\right)^2 \qquad \boxed{P_{cd} = 0.667} \text{ klf}$$

$$P_{ch} := \frac{hau \cdot w_{tf}}{12^2} \cdot w_c \qquad \boxed{P_{ch} = 0.100} \text{ klf}$$

$$P_{cp} := w_p \qquad \boxed{P_{cp} = 0.129} \text{ klf}$$

$$P_{cfws} := w_{ws} \qquad \boxed{P_{cfws} = 0.133} \text{ klf}$$

$$P_c := P_{cd} + P_{ch} + P_{cp} + P_{cfws} \qquad \boxed{P_c = 1.029} \text{ klf}$$

From earlier calculations, the maximum #4 stirrup spacing used is $s = 18.0$ inches.

$$A_{vf} := \frac{A_v}{s} \cdot 12 \qquad \boxed{A_{vf} = 0.267} \text{ in}^2/\text{ft}$$

$$V_n := c \cdot A_{cv} + \mu \cdot \left(A_{vf} \cdot f_y + P_c\right) \qquad \boxed{V_n = 77.5} \text{ kips/ft}$$

$$V_{n1} := K_1 \cdot f'_{cd} \cdot A_{cv} \qquad \boxed{V_{n1} = 259.2} \text{ kips/ft}$$

$$V_{n2} := K_2 \cdot A_{cv} \qquad \boxed{V_{n2} = 388.8} \text{ kips/ft}$$

$$V_n := \min\left(V_n, V_{n1}, V_{n2}\right) \qquad \boxed{V_n = 77.5} \text{ kips/ft}$$

$V_r := \phi_v \cdot V_n$ $\boxed{V_r = 69.8}$ kips/ft

$\boxed{V_{ui} = 65.4}$ kips/ft

Is V_r greater than V_{ui}? $\boxed{check = \text{"OK"}}$

Solution:
#4 stirrups spaced at $s = 18.0$ inches is adequate to develop the required interface shear resistance for the entire length of the girder.

E19-1.16 Deflection Calculations

Check the Live Load deflection with the vehicle loading as specified in **LRFD [3.6.1.3.2]**; design truck alone or 25% of the design truck + the lane load.

The deflection shall be limited to L/800.

The moment of inertia of the entire bridge shall be used.

$\Delta_{limit} := \frac{L \cdot 12}{800}$ $\boxed{\Delta_{limit} = 2.190}$ inches

$I_{cg} = 1203475.476$

$ng = 6$ number of girders

$I_{bridge} := I_{cg} \cdot ng$ $\boxed{I_{bridge} = 7220853}$ in^4

From CBA analysis with 3 lanes loaded, the truck deflection controlled:

$\Delta_{truck} := 0.648$ in

Applying the multiple presence factor from **LRFD Table [3.6.1.1.2-1]** for 3 lanes loaded:

$\Delta := 0.85 \cdot \Delta_{truck}$ $\boxed{\Delta = 0.551}$ in

Is the actual deflection less than the allowable limit, $\Delta < \Delta$ limit? $\boxed{check = \text{"OK"}}$

E19-1.17 Camber Calculations

Moment due to straight strands:

Number of straight strands:	$ns_s = 38$	
Eccentricity of the straight strands:	$y_s = -30.66$	in
$P_{i\_s} := ns_s \cdot A_s \cdot \left(f_{tr} - \Delta f_{pES}\right)$	$P_{i\_s} = 1520$	kips
$M_1 := P_{i\_s} \cdot \left\|y_s\right\|$	$M_1 = 46598$	kip-in

Upward deflection due to straight strands:

Length of the girder:	$L_g = 147$	ft
Modulus of Elasticity of the girder at release:	$E_{ct} = 4999$	ksi
Moment of inertia of the girder:	$I_g = 656426$	in⁴
$\Delta_s := \frac{M_1 \cdot L_g^{\,2}}{8 \cdot E_{ct} \cdot I_g} \cdot 12^2$	$\Delta_s = 5.523$	in

Moment due to draped strands:

$P_{i\_d} := ns_d \cdot A_s \cdot \left(f_{tr} - \Delta f_{pES}\right)$	$P_{i\_d} = 319.971$	kips
	$A = 67.000$	in
	$C = 5.000$	in
$M_2 := P_{i\_d} \cdot (A - C)$	$M_2 = 19838.175$	kip-in
$M_3 := P_{i\_d} \cdot \left(A - \left\|y_b\right\|\right)$	$M_3 = 10280.654$	kip-in

Upward deflection due to draped strands:

$\Delta_d := \frac{L_g^{\,2}}{8 \cdot E_{ct} \cdot I_g} \cdot \left(\frac{23}{27} \cdot M_2 - M_3\right) \cdot 12^2$	$\Delta_d = 0.784$	in

Total upward deflection due to prestress:

$\Delta_{PS} := \Delta_s + \Delta_d$	$\Delta_{PS} = 6.308$	in

For flexure in non-prestressed concrete, $\phi_f := 0.9$.

The width of the bottom flange of the girder, $b_w = 30.00$ inches.

$$R_u := \frac{M_u \cdot 12}{\phi_f \cdot b_w \cdot d_e^2}$$ $\boxed{R_u = 0.532}$ ksi

$$\rho := 0.85 \frac{f'_c}{f_y} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot R_u}{0.85 \cdot f'_c}}\right)$$ $\boxed{\rho = 0.00925}$

$$A_s := \rho \cdot b_w \cdot d_e$$ $\boxed{A_s = 16.74}$ in^2

This reinforcement is distributed over the effective flange width calculated earlier, $w_e = 90.00$ inches. The required continuity reinforcement in in^2/ft is equal to:

$$As_{req} := \frac{A_s}{\frac{w_e}{12}}$$ $\boxed{As_{req} = 2.232}$ in^2/ft

From Chapter 17, Table 17.5-3, for a girder spacing of $S = 7.5$ feet and a deck thickness of $t_s = 8.0$ inches, use a longitudinal bar spacing of #4 bars at $s_{longit} := 8.5$ inches. The continuity reinforcement shall be placed at 1/2 of this bar spacing, .

#9 bars at 4.25 inch spacing provides an $\boxed{As_{prov} = 2.82}$ in^2/ft, or the total area of steel provided:

$$As := As_{prov} \cdot \frac{w_e}{12}$$ $\boxed{As = 21.18}$ in^2

Calculate the capacity of the section in flexure at the pier:

Check the depth of the compression block:

Assume $f_s = f_y$

$$a := \frac{As \cdot f_y}{0.85 \cdot b_w \cdot f'_c}$$ $\boxed{a = 6.228}$ in

This is within the thickness of the bottom flange height of 7.5 inches.

If $\frac{c}{d_s} \le 0.6$ for ($f_y = 60$ ksi) **LRFD [5.7.2.1]**, the reinforcement has yielded and the assumption is correct.

$\beta_1 := 0.85$ $$c := \frac{a}{\beta_1}$$ $\boxed{c = 7.327}$ in

$\frac{c}{d_s} = 0.12 < 0.6$ therefore, the reinforcement will yield

$$M_n := As \cdot f_y \cdot \left(d_e - \frac{a}{2}\right) \cdot \frac{1}{12}$$ $\boxed{M_n = 6056}$ kip-ft

$$M_r := \phi_f \cdot M_n$$ $\boxed{M_r = 5451}$ kip-ft

$\boxed{M_u = 4358}$ kip-ft

Is M_u less than M_r? $\boxed{check = \text{"OK"}}$

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

$$f_r := 0.24 \cdot \sqrt{f'_{cd}}$$ $\boxed{f_r = 0.480}$ ksi

$$M_{cr} = \gamma_3 (\gamma_1 \cdot f_r) S_c$$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement for A615, Grade 60 reinforcement

$$M_{cr} := 1.1 f_r \cdot S_c \cdot \frac{1}{12}$$ $\boxed{M_{cr} = 1709}$ kip-ft

$\boxed{1.33 \cdot M_u = 5796}$ kip-ft

Is M_r greater than the lesser value of M_{cr} and $1.33*M_u$? $\boxed{check = \text{"OK"}}$

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

$$\rho := \frac{As}{b_w \cdot d_e}$$ $\boxed{\rho = 0.01170}$

$$n := \frac{E_s}{E_B}$$ $\boxed{n = 4.566}$

$$k := \sqrt{(\rho \cdot n)^2 + 2 \cdot \rho \cdot n} - \rho \cdot n$$ $\boxed{k = 0.278}$

$$j := 1 - \frac{k}{3}$$ $\boxed{j = 0.907}$

$$\Delta f := n \cdot \frac{\left|M_{LLfatigue}\right| \cdot y_{rb}}{I_{cr}} \cdot 12$$ $\boxed{\Delta f = 3.488}$ ksi

$\boxed{\gamma f_{LL} \cdot \Delta f = 5.232}$ ksi

Is $\gamma f_{LL} \cdot \Delta f$ less than ΔF_{TH}? $\boxed{check = \text{"OK"}}$

E19-2.13 Bar Cut Offs

The first cut off is located where half of the continuity reinforcement satisfies the moment diagram. Non-composite moments from the girder and the deck are considered along with the composite moments when determining the Strength I moment envelope. (It should be noted that since the non-composite moments are opposite in sign from the composite moments in the negative moment region, the minimum load factor shall be applied to the non-composite moments.) Only the composite moments are considered when checking the Service and Fatigue requirements.

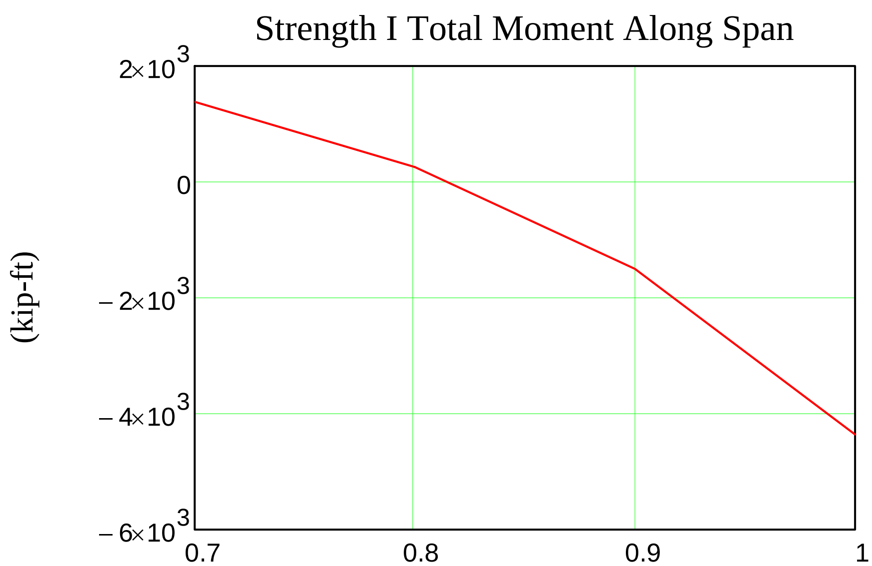
$$spa' := spa \cdot 2$$ $\boxed{spa' = 8.50}$ in

$$As' := \frac{As}{2}$$ $\boxed{As' = 10.588}$ in^2

$$a' := \frac{As' \cdot f_y}{0.85 \cdot b_w \cdot f'_c}$$ $\boxed{a' = 3.11}$ in

$$M_{n'} := As' \cdot f_y \cdot \left(d_e - \frac{a'}{2}\right) \cdot \frac{1}{12}$$ $\boxed{M_{n'} = 3111}$ kip-ft

$M_{r'} := \phi_f \cdot M_{n'}$ $\boxed{M_{r'} = 2799}$ kip-ft



Based on the moment diagram, try locating the first cut off at $cut_1 := 0.90$ span. Note that the Service I crack control requirements control the location of the cut off.

$\boxed{M_{r'} = 2799}$ kip-ft

$\boxed{Mu_{cut1} = 1501}$ kip-ft

$\boxed{Ms_{cut1} = 1565}$ kip-ft

Is Mu_{cut1} less than $M_{r'}$? $\boxed{check = \text{"OK"}}$

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

$\boxed{M_{cr} = 1709}$ kip-ft

E19-3.11 Flexural Capacity at Midspan

Check f_{pe} in accordance with **LRFD [5.7.3.1.1]**:

$\boxed{f_{pe} = 172}$ ksi $\boxed{0.5 \cdot f_{pu} = 135}$ ksi

Is $0.5*f_{pu}$ less than f_{pe}? $\boxed{check = \text{"OK"}}$

Then at failure, we can assume that the tendon stress is:

$$f_{ps} = f_{pu}\left(1 - k \cdot \frac{c}{d_p}\right)$$

where:

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right)$$

From **LRFD Table [C5.7.3.1.1-1]**, for low relaxation strands, $k := 0.28$.

"c" is defined as the distance between the neutral axis and the compression face (inches).

Assume that the compression block is in the top section of the box. Calculate the capacity as if it is a rectangular section. The neutral axis location,calculated in accordance with **LRFD 5.7.3.1.1** for a rectangular section, is:

$$c = \frac{A_{ps} \cdot f_{pu}}{0.85 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$

where:

$A_{ps} := N \cdot A_s$ $\boxed{A_{ps} = 2.45}$ in^2

$b := W_s \cdot 12$ $\boxed{b = 48.00}$ in

LRFD [5.7.2.2]

$\beta_1 := \max\left[0.85 - \left(f'_c - 4\right) \cdot 0.05, 0.65\right]$ $\boxed{\beta_1 = 0.800}$

$d_p := y_t - e_s$ $\boxed{d_p = 18.75}$ in

$$c := \frac{A_{ps} \cdot f_{pu}}{0.85 \cdot f'_c \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c = 3.82}$ in

$a := \beta_1 \cdot c$ $\boxed{a = 3.06}$ in

This is within the depth of the top slab (5-inches). Therefore our assumption is OK.

Now calculate the effective tendon stress at ultimate:

$$f_{ps} := f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right) \qquad \boxed{f_{ps} = 254.6} \quad \text{ksi}$$

$$T_u := f_{ps} \cdot A_{ps} \qquad \boxed{T_u = 624} \quad \text{kips}$$

Calculate the nominal moment capacity of the section in accordance with **LRFD [5.7.3.2]**:

$$M_n := \left[A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right)\right] \cdot \frac{1}{12}$$

$$\boxed{M_n = 895} \quad \text{kip-ft}$$

For prestressed concrete, $\phi_f := 1.00$, **LRFD [5.5.4.2.1]**. Therefore the usable capacity is:

$$M_r := \phi_f \cdot M_n \qquad \boxed{M_r = 895} \quad \text{kip-ft}$$

The required capacity:

Exterior Girder Moment

$$M_u := M_{str} \qquad \boxed{M_u = 862} \quad \text{kip-ft}$$

Check the section for minimum reinforcement in accordance with **LRFD [5.7.3.3.2]** for the interior girder:

$$\boxed{1.33 \cdot M_u = 1147} \quad \text{kip-ft}$$

$f_r := 0.24 \cdot \sqrt{f'_c}$ **LRFD [5.4.2.6]** $\qquad \boxed{f_r = 0.537}$ ksi

$$f_{cpe} := \frac{T}{A} + \frac{T \cdot e_s}{S_b} \qquad \boxed{f_{cpe} = 1.816} \quad \text{ksi}$$

$$S_c := -S_b \qquad \boxed{S_c = 3137} \quad \text{ksi}$$

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_2 := 1.1$ prestress variability factor

$\gamma_3 := 1.0$ for prestressed concrete structures

$$M_{cr} := \gamma_3 \cdot \left[S_c \cdot \left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe}\right) \cdot \frac{1}{12}\right]$$ $\boxed{M_{cr} = 747}$ kip-ft

Is M_r greater than the lesser value of M_{cr} and 1.33*M_u? $\boxed{check = \text{"OK"}}$

E19-3.12 Shear Analysis

A separate analysis must be conducted to estimate the total shear force in each girder for shear design purposes.

The live load shear distribution factors to the girders are calculated above in E19-3.2.2.

$\boxed{g_{int\_v} = 0.600}$

$\boxed{g_{ext\_v} = 0.744}$

From section E19-3.4, the uniform dead loads on the girders are:

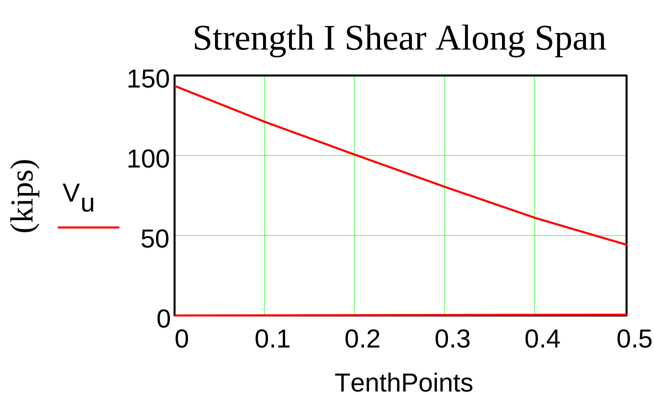
Interior Girder $\boxed{w_{DCint} = 0.792}$ klf

$\boxed{w_{DWint} = 0.082}$ klf

Exterior Girder $\boxed{w_{DCext} = 0.845}$ klf

$\boxed{w_{DWext} = 0.083}$ klf

However, the internal concrete diaphragms were applied as total equivalent uniform loads to determine the maximum mid-span moment. The diaphragm weights should be applied as poin loads for the shear calculations.



$\boxed{Vu_{0.0} = 143.5}$ kips

$\boxed{Vu_{0.5} = 44.2}$ kips

Simplified Procedure for Prestressed and Nonprestressed Sections, **LRFD [5.8.3.4.3]**

$b_v := 2t_w$ $\boxed{b_v = 10.00}$ in

The critical section for shear is taken at a distance of d_v from the face of the support, **LRFD [5.8.3.2]**.

d_v = effective shear depth taken as the distance between the resultants of the tensile and compressive forces due to flexure. It need not be taken less than the greater of 0.9*d_e or 0.72h (inches). **LRFD [5.8.2.9]**

The first estimate of d_v is calculated as follows:

$d_v := -e_s + y_t - \frac{a}{2}$ $\boxed{d_v = 17.22}$ in

For the standard bearing pad of width, $w_{brg} := 8$ inches, the distance from the end of the girder to the critical section:

$L_{crit} := \left(w_{brg} + d_v\right) \cdot \frac{1}{12}$ $\boxed{L_{crit} = 2.10}$ ft

The eccentricity of the strand group at the critical section is:

$\boxed{e_s = -8.25}$ in

Calculation of compression stress block:

$\boxed{d_p = 18.75}$ in

$\boxed{A_{ps} = 2.45}$ in^2

Also, the value of f_{pu}, should be revised if the critical section is located less than the development length from the end of the beam. The development length for a prestressing strand is calculated in accordance with **LRFD [5.11.4.2]**:

$K := 1.0$ for prestressed members with a depth less than 24 inches

$\boxed{d_s = 0.5}$ in

$l_d := K \cdot \left(f_{ps} - \frac{2}{3} \cdot f_{pe}\right) \cdot d_s$ $\boxed{l_d = 70.0}$ in

The transfer length may be taken as: $l_{tr} := 60 \cdot d_s$ $\boxed{l_{tr} = 30.00}$ in

Since $L_{crit} = 2.102$ feet is between the transfer length and the development length, the design stress in the prestressing strand is calculated as follows:

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vertical. Also, inspection manholes are often inserted in the bottom flanges of closed-box sections near supports. These manholes should be subtracted from the bottom-flange area when computing the elastic section properties for use in the region of the access hole. If longitudinal flange stiffeners are present on the closed-box section, they are often included when computing the elastic section properties.

When investigating web bend-buckling resistance for closed-box sections, **LRFD [6.11.3.2]** states that the maximum longitudinal flange stress due to the factored loads, calculated without consideration of longitudinal warping, must not exceed $\phi_f F_{crw}$ at sections where non-composite box flanges are subject to compression during construction. For more information about the web bend-buckling resistance of box girders, refer to 24.12.1. In *AASHTO LRFD*, a box flange is defined as a flange connected to two webs.

Torsion in structural members is generally resisted through a combination of St. Venant torsion and warping torsion. For closed cross-sections such as box girders, St. Venant torsion generally dominates. Box girders possess favorable torsional characteristics which make them an attractive choice for horizontally curved bridges. However, due to redundancy concerns, use of single-box and two-box girder bridges should be avoided unless absolutely necessary.

WisDOT policy item:

Rigorous analysis of single-box and two-box girder bridges to eliminate the need for in-depth fracture critical inspections is not allowed.

24.16 Design Examples

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27.1 General

Bridges supported in the conventional way by abutments and piers require bearings to transfer girder reactions without overstressing the supports, ensuring that the bridge functions as intended. Bridges usually require bearings that are more elaborate than those required for building columns, girders and trusses. Bridge bearings require greater consideration in minimizing forces caused by temperature change, friction and restraint against elastic deformations. A more detailed analysis in bridge bearing design considers the following:

- Bridges are usually supported by reinforced concrete substructure units, and the magnitude of the horizontal thrust determines the size of the substructure units. The coefficient of friction on bridge bearings should be as low as possible.

- Bridge bearings must be capable of withstanding and transferring dynamic forces and the resulting vibrations without causing eventual wear and destruction of the substructure units.

- Most bridges are exposed to the elements of nature. Bridge bearings are subjected to more frequent and greater total expansion and contraction movement due to changes in temperature than those required by buildings. Since bridge bearings are exposed to the weather, they are designed as maintenance-free as possible.

WisDOT policy item:

WisDOT uses an installation temperature of 60°F for designing bearings. The temperature range considered for prestressed concrete girder superstructures is 5°F to 85°F, resulting in a range of 60° - 5° = 55° for bearing design. For prestressed girders an additional shrinkage factor of 0.0003 ft/ft should also be accounted for. The temperature range considered for steel girder superstructures is -30°F to 120°F, resulting in a range of 60° - (-30°) = 90° for bearing design.

WisDOT policy item:

According to **LRFD [14.4.1]**, the influence of dynamic load allowance need not be included for bearings. However, dynamic load allowance shall be included when designing bearings for bridges in Wisconsin. Apply dynamic load allowance in **LRFD [3.6.2]** to HL-93 live loads as stated in **LRFD [3.6.1.2, 3.6.1.3]** and distribute these loads, along with dead loads, to the bearings.

27.4 Design Example

E27-1 Steel Reinforced Elastomeric Bearing

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30.1 Crash-Tested Bridge Railings and FHWA Policy

Crash test procedures for full-scale testing of guardrails were first published in 1962 in the *Highway Research Correlation Services Circular 482*. This was a one-page document that specified vehicle mass, impact speed, and approach angle for crash testing and was aimed at creating uniformity to traffic barrier research between several national research agencies.

In 1974, the National Cooperative Highway Research Program (NCHRP) published their final report based on NCHRP Project 22-2, which was initiated to address outstanding questions that were not covered in *Circular 482*. The final report, NCHRP Report 153 – *"Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances,"* was widely accepted following publication; however, it was recognized that periodic updating would be required.

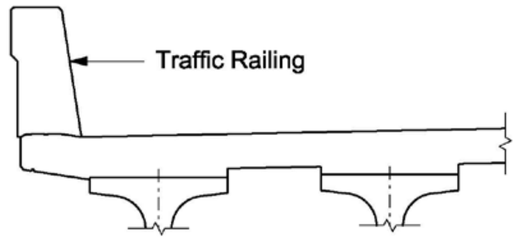
NCHRP Project 22-2(4) was initiated in 1979 to address major changes to reflect current technologies of that time and NCHRP Report 230, *"Recommended Procedures for the Safety Performance Evaluation of Highway Safety Appurtenances,"* was published in 1980. This document became the primary reference for full-scale crash testing of highway safety appurtenances in the U.S. through 1993.

In 1986, the Federal Highway Administration (FHWA) issued a policy memorandum that stated highway bridges on the National Highway System (NHS) and the Interstate Highway System (IHS) must use crash-tested railings in order to receive federal funding.

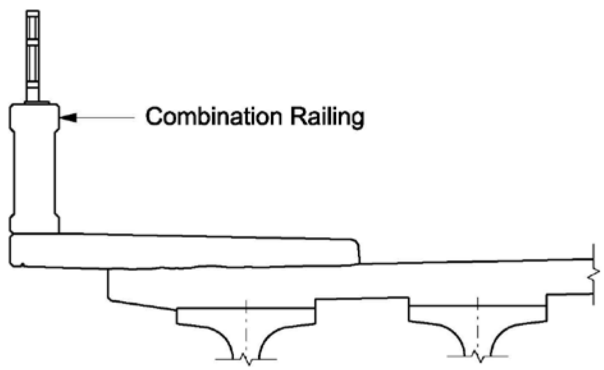
The American Association of State Highway and Transportation Officials (AASHTO) recognized that the evolution of roadside safety concepts, technology, and practices necessitated an update to NCHRP Report 230 approximately 7 years after its adoption. NCHRP Report 350, *"Recommended Procedures for the Safety Performance Evaluation of Highway Features,"* represented a major update to the previously adopted report. The updates were based on significant changes in the vehicle fleet, the emergence of many new barrier designs, increased interest in matching safety performance to levels of roadway utilization, new policies requiring the use of safety belts, and advances in computer simulation and other evaluation methods.

NCHRP Report 350 differs from NCHRP Report 230 in the following ways: it is presented in all-metric documentation, it provides a wider range of test procedures to permit safety performance evaluations for a wider range of barriers, it uses a pickup truck as the standard test vehicle in place of a passenger car, it defines other supplemental test vehicles, it includes a broader range of tests to provide a uniform basis for establishing warrants for the application of roadside safety hardware that consider the levels of use of the roadway facility, it includes guidelines for selection of the critical impact point for crash tests on redirecting-type safety hardware, it provides information related to enhanced measurement techniques related to occupant risk, and it reflects a critical review of methods and technologies for safety-performance evaluation.

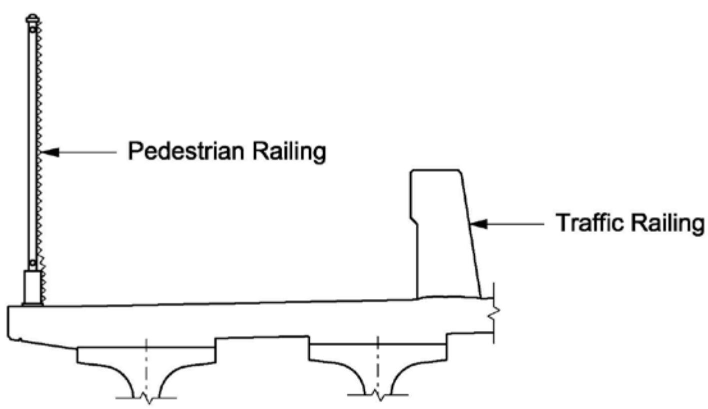
In May of 1997, a memorandum from Dwight A. Horne, the FHWA Chief of the Federal-Aid and Design Division, on the subject of "Crash Testing of Bridge Railings" was published. This memorandum identified 68 crash-tested bridge rails, consolidated earlier listings, and



Traffic Railing
All Design Speeds



Combination Railling
Design Speeds of 45 mph or Less



Pedestrian Railing
All Design Speeds

Figure 30.2-1
Bridge Railing Types

The application of bridge railings shall comply with the following guidance:

1. All bridge railings shall conform to **LRFD [13]**.

2. Traffic Railings placed on state-owned and maintained structures (Interstate Highways, United States Highways, State Trunk Highways) with a design speed exceeding 45 mph shall be solid concrete parapets. Where the minimum 0.5% deck grade cannot be accommodated for proper drainage based on project specific constraints, designer shall utilize open railings as described in this section.

   Traffic Railings placed on locally-owned and maintained structures (County Trunk Highways, Local Roadways) with a design speed exceeding 45 mph are strongly encouraged to utilize solid concrete parapets.

3. Traffic Railings placed on structures with a design speed of 45 mph or less can be either solid concrete parapets or open railings with the exception as noted below in the single slope parapet application section.

4. New bridge plans utilizing concrete parapets shall be designed with “SS” (“32SS”, “36SS”, “42SS”, or “56SS”) parapets.

5. Per **LRFD [13.7.3.2]**, the minimum Traffic Railing height shall be 27” for TL-3, 32” for TL-4, 42” for TL-5, and 90” for TL-6. The railing applications as noted in this section meet these requirements.

6. Per **LRFD [13.8.1]** and **LRFD [13.9.2]**, the minimum height of a Pedestrian (and/or bicycle) Railing shall be 42” measured from the top of the walkway or riding surface respectively. Per the *Wisconsin Bicycle Facility Design Handbook*, on bridges that are signed or marked as bikeways and bicyclists are operating right next to the railing, the preferred height of the railing is 54”. The higher railing/parapet height is especially important and should be used on long bridges, high bridges, and bridges having high bicyclist volumes. If an open railing is used, the clear opening between horizontal elements shall be 6 inches or less.

7. Aesthetics associated with bridge railings shall follow guidance provided in Section 30.4.

The designation for railing types are shown on the Standard Details. Bridge railings shall be employed as follows:

1. The single slope parapet “32SS” shall be used as a Traffic Railing on all structures with a design speed exceeding 45 mph. The “36SS” and “42SS” parapets should be used where the Region determines that there is high truck traffic and/or curved horizontal alignment creating more potential for overtopping the parapet, or if roadway concrete barrier single slope (CBSS) of the same height adjoins the bridge. Single slope parapet “56SS” shall only used if 56” CBSS adjoins the bridge. The “SS” parapets were crash-tested per NCHRP Report 350 specifications and meet crash test criteria for TL-4.

A “SS” or solid parapet shall be used on all grade separation structures and railroad crossings to minimize snow removal falling on the traffic below.

2. The sloped face parapet "LF" and “HF” parapets shall be used as Traffic Railings for rehabilitation projects only to match the existing parapet type. The sloped face parapets were crash-tested per NCHRP Report 230 and meet NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

3. The “51F” parapet shall only be used as a Traffic Railing on the median side of a structure when it provides a continuation of an approach 51 inch high median barrier.

4. The vertical face parapet “A” can be used for all design speeds. The vertical face parapet is recommended for use as a Combination Railing on raised sidewalks or as a Traffic Railing where the design speed is 45 mph or less. If the structure has a raised sidewalk on one side only, a sloped parapet should be used on the side opposite of the sidewalk. For design speeds exceeding 45 mph, at locations where the parapet is protected by a Traffic Railing between the roadway and a sidewalk at grade, the vertical face parapet can be used as a Pedestrian Railing. Under some circumstances, the vertical face parapet “A” can be used as a Traffic Railing for design speeds exceeding 45 mph with the approval of the Bureau of Structures Development Section. The vertical face parapet “A” was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

5. Aesthetic railings may be used if crash tested according to Section 30.1 or follow the guidance provided in Section 30.4.

   The Texas style aesthetic parapet, type “TX”, can be used as a Combination Railing or Traffic Railing on raised sidewalks on structures with a design speed of 45 mph or less. For design speeds exceeding 45 mph, at locations where the parapet is protected by a Traffic Railing between the roadway and a sidewalk at grade, the type “TX” parapet can be used. This parapet is very expensive; however, form liners simulating the openings can be used to reduce the cost of this parapet. The type “TX” parapet was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-2 based on a May 1997 FHWA memorandum.

6. The type “PF” tubular railing, as shown in the Standard Details, shall not be used on bridge plans with a PS&E after 2013. This railing was not allowed on the National Highway System (NHS). The type “PF” railing was used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less.

7. Combination Railings, type “C1” through “C6”, are shown in the Standard Details and are approved as aesthetic railings attached to concrete parapets. The aesthetic additions are placed at least 5” from the crash-tested rail face per the Standard Details and have previously been determined to not present a snagging potential. These railings can only be used when the design speed is 45 mph or less, or the railing is protected by a Traffic Railing between the roadway and a sidewalk at grade. The crash test criteria of the combination railings are based on the concrete parapets

to which they are attached (i.e., if a type “C1” combination railing is attached to the top of a vertical face parapet type “A”, the railing meets crash test criteria for TL-4).

8. Chain Link Fence and Ornamental Protective Screening, as shown in the Standard Details, may be attached to the top of concrete parapets as part of a Combination Railing or as a Pedestrian Railing attached directly to the deck if protected by a Traffic Railing between the roadway and a sidewalk at grade. Chain Link Fence, when attached to the top of a concrete Traffic Railing, can be used for design speeds exceeding 45 mph. Due to snagging and breakaway potential of the vertical spindles, Ornamental Protective Screening should only be used when the design speed is 45 mph or less, or the screening is protected by a Traffic Railing between the roadway and a sidewalk at grade.

9. Type "H" aluminum or steel railing can be used on top of either vertical face or single slope parapets (“A” or “SS”) as part of a Combination Railing when required for pedestrians and/or bicyclists. For a design speed greater than 45 mph, the single slope parapet is recommended. Per the Standard Specifications, the contractor shall furnish either aluminum railing or steel railing. In general, the bridge plans shall include both options. For a specific project, one option may be required. This may occur when rehabilitating a railing to match an existing railing or when painting of the railing is required (requires steel option). If one option is required, the designer shall place the following note on the railing detail sheet: “Type H (insert railing type) railing shall not be used”. The combination railing was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

10. Timber Railing as shown in the Standard Details is not allowed on the National Highway System (NHS). Timber Railing may be used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less. The Timber Railing was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-2 based on a May 1997 FHWA memorandum.

11. The type "W" railing, as shown in the Standard Details, is not allowed on the National Highway System (NHS). This railing may be used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less. The type “W” railing shall be used on concrete slab structures only. The use of this railing on girder type structures has been discontinued. Generally, type "W" railing is considered when the roadway approach requires standard beam guard and if the structure is 80 feet or less in length. The type “W” railing was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-3 based on a May 1997 FHWA memorandum.

12. Type “M” steel railing, as shown in the Standard Details, shall generally be used as a Traffic Railing on all functional classes of Wisconsin structures with a design speed of 45 mph or less. The type “M” railing may be used on roadways with a design speed exceeding 45 mph where the minimum 0.5% deck grade cannot be accommodated for proper drainage based on project specific constraints. The type “M” railing also can be used in place of the type “W” railing when placed on girder type structures as type “W” railings are not allowed for this application. However, the type “M” railing is

not allowed for use on prestressed box girder bridges. This railing shall be considered where the Region requests an open railing. The type “M” railing was crash-tested per NCHRP Report 350 and meets criteria for TL-4.

13. The type "F" steel railing, as shown in the Standard Details, shall not be used on new bridge plans with a PS&E after 2013. It has not been allowed on the National Highway System (NHS) in the past and was used on non-NHS roadways with a design speed of 45 mph or less.

14. If a box culvert has a Traffic Railing across the structure, then the railing members shall have provisions for a thrie beam connection at the ends of the structure as shown in the *Facilities Development Manual (FDM) Standard Detail Drawings (SDD) 14b20*. Railing is not required on box culverts if the culvert is extended to provide an adequate clear zone as defined in *FDM 11-15-1*. Non-traversable hazards or fixed objects should not be constructed or allowed to remain within the clear zone. When this is not feasible, the use of a Traffic Railing to shield the hazard or obstacle may be warranted. The railing shall be provided only when it is cost effective as defined in *FDM Procedure 11-45-1*.

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

<u>*Note: WisDOT is currently investigating alternative open railing types for future use on bridge structures in Wisconsin. Specifically, new rail standards to replace the existing type “W” and type “M” railings are being considered.*</u>

See the *FDM* for additional railing application requirements. See *11-45-1* and *11-45-2* for Traffic Barrier, Crash Cushions, and Roadside Barrier Design Guidance. See *11-35-1*, *Table 1.2* for requirements when barrier wall separation between roadway and sidewalk is necessary.

30.3 General Design Details

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

2. Adhesive anchored parapets are allowed at interior Traffic Railing locations only when the adjacent exterior parapet is a crash test approved Traffic Railing per Section 30.2 (i.e., cast-in-place anchors are used at exterior parapet location). See Standard Details 30.10 and 30.14 for more information.

3. Sign structures, sign trusses, and monotubes shall be placed on top of railings to meet the working width and zone of intrusion dimensions noted in *FDM 11-45 Section 2.3.6.2.2* and *Section 2.3.6.2.3* respectively.

4. It is desirable to avoid attaching noise walls to bridge railings. However, in the event that noise walls are required to be located on bridge railings, compliance with the setback requirements stated in Section 30.4 and what is required in *FDM 11-45 Sections 2.3.6.2.2* and *2.3.6.2.3* is not required. *Note: WisDOT is currently investigating the future use of noise walls on bridge structures in Wisconsin.*

5. Temporary bridge barriers shall be designed in accordance with *FDM SDD 14b7*. Where temporary bridge barriers are being used for staged construction, the designer should attempt to meet the required offsets so that the barrier does not require anchorage which would necessitate drilling holes in the new deck.

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

7. Refer to Standard Detail 30.07 – Vertical Face Parapet "A" – for detailing concrete parapet or sidewalk deflection joints. These joints are used based on previous experience with transverse deck cracking beneath the parapet joints.

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

9. For beam guard type "W" railing, locate the expansion splice at a post or on either side of the expansion joint.

10. Sidewalks - If there is a Traffic Railing between the roadway and an at grade sidewalk, and the roadway side of the Traffic Railing is more than 11'-0" from the exterior edge of deck, access must be provided to the at grade sidewalk for the snooper truck to inspect the underside of the bridge. The sidewalk width must be

30.5 Utilities

The maximum allowable conduits that can be placed in “SS”, “LF” or “HF” parapets are shown in the following sketches (“32SS” shown). Junction (Pull) boxes can only be used with 2 inch diameter conduit. The maximum length of 3 inch conduit is 190 feet, as no boxes are allowed.

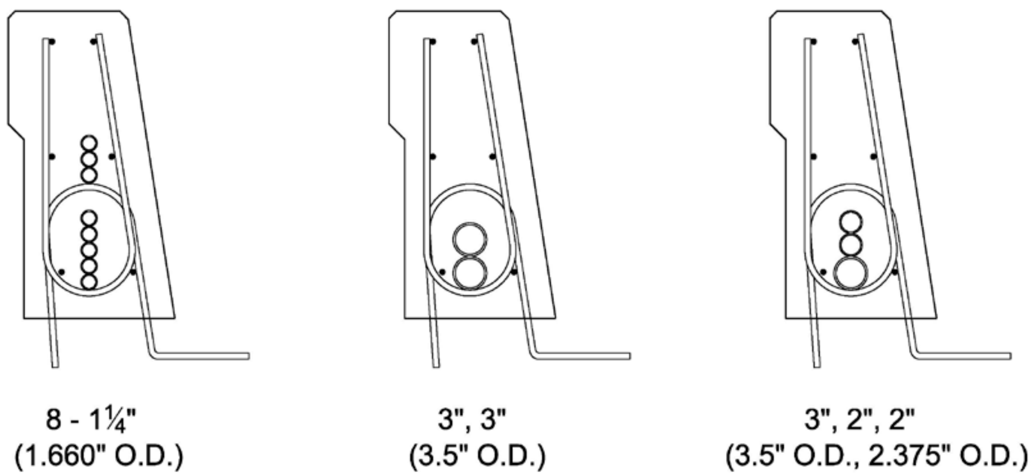


Figure 30.5-1
Maximum Allowable Conduits in “SS”, “LF” and “HF” Parapets

When light poles are mounted on top of parapets and the design speed exceeds 45 mph, the light pole must be located behind the back edge of the parapet. See Standard Detail 30.21 – Light Standard, Junction Box, & Expansion Fitting for “SS” Parapets – for typical light pole blister detail information. The poles should also be placed over the piers unless there is an expansion joint at that location. If an expansion joint is present, place 4 feet away. *FDM 9-25-5* addresses whether a bench mark disk should be set on a structure; however, structures are not usually preferred due to possible elevation changes from various causes. See Bridge Manual section 6.3.3.7 for more information regarding bench mark disks.

30.6 Protective Screening

Protective screening is a special type of fence constructed on the sides of an overpass to discourage and/or prevent people from dropping or throwing objects onto vehicles passing underneath the structure. Protective screening is generally chain link type fencing attached to steel posts mounted on top of a Traffic Railing (part of a Combination Railing) or on a sidewalk surface (Pedestrian Railing). The top of the protective screening may be bent inward toward the roadway, if mounted on a Traffic Railing and on a raised sidewalk, to prevent objects from being thrown off the overpass structure. The top of the protective screening may also be bent inward toward the sidewalk, if mounted directly to the deck when it is protected by a Traffic Railing between the roadway and a sidewalk at grade. Aesthetics are enhanced by using a colored protective screening which can be coordinated with the color of the structure. See Chapter 30 and Chapter 37 Standard Details for protective screening detail information.

Examples of situations that warrant consideration of protective screening are:

1. Location with a history of, or instances of, objects being dropped or thrown from an existing overpass.

2. All new overpasses if there have been instances of objects being dropped or thrown at other existing overpasses in the area.

3. Overpasses near schools, playgrounds, residential areas or any other locations where the overpass may be used by children who are not accompanied by an adult.

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

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.

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

Protective screening (or Pedestrian Railing) may be required for particular structures based on the safety requirements of the users on the structure and those below. Roadway designers, bridge designers, and project managers should coordinate this need and relay the information to communities involved when aesthetic details are being formalized.

See *FDM 11-35-1.8* for additional guidance pertaining to protective screening usage requirements.

Occasionally, access to light poles behind protective screening is required or the screening may need repair. To gain access, attach fence stretchers to the fencing and remove one vertical wire by threading or cutting. To repair, attach fence stretchers and thread a vertical wire in place of the one removed by either reusing the one in place or using a new one.

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

See Section 30.3 for additional guidance with regards to snooper truck access, screening height, and straight vs. bent fencing.

30.7 Medians

The typical height of any required median curb is 6 inches. This will prevent normal crossovers and reduce vaulting on low speed roadways without excessive dead load being applied to the superstructure. On structure rehabilitations, the height of median may increase up to 8” to match the existing median at the bridge approaches. Contact the Bureau of Structures Development Section if median heights in excess of 8 inches are desired. See Standard Detail 17.01 – Median and Raised Sidewalk Details – for typical raised median detail information.

30.8 Railing Rehabilitation

The FHWA, in its implementation plan for MASH, requires that bridge railings on the NHS shall meet the requirements of MASH or NCHRP Report 350. In addition, FHWA states that "Agencies are encouraged to upgrade existing highway safety hardware that has not been accepted under MASH or NCHRP Report 350 during reconstruction projects, during 3R (Resurfacing, Restoration, Rehabilitation), or when the railing system is damaged beyond repair".

WisDOT requirements for the treatment of existing railings for various project classifications are outlined in Table 30.8-1:

Project Classification	**Railing Rehabilitation**
Preventative Maintenance✳ (Resurfacing, Restoration)	Replacement of bridge railing not in compliance with MASH or NCHRP Report 350 is recommended but not required. Existing railings – both in compliance and not in compliance – with MASH, NCHRP Report 350, and NCHRP Report 230 may be altered to improve the performance of the existing railings where it is not feasible to install an approved railing. Coordination with BOS and BPD is required. <u>NHS Structures</u>: It is strongly encouraged that existing railing that does not comply with MASH, NCHRP Report 350, or NCHRP Report 230 be upgraded to comply with MASH or NCHRP Report 350. <u>Non-NHS Structures</u>: It is strongly encouraged that existing railing that does not comply with MASH, NCHRP Report 350, or NCHRP Report 230 be upgraded to comply with MASH or NCHRP Report 350.

<table>
<tr><td rowspan="2">3R✳✳ (Resurfacing, Restoration, Rehabilitation)</td><td>If rehabilitation work, as part of the 3R project, is scheduled or performed which does not widen the structure nor affect the existing railing.</td><td>Replacement of bridge railing not in compliance with MASH or NCHRP Report 350 is recommended but not required provided the minimum rail height requirement is met. (Minimum rail height shall be 27” for roadway design speed of 45 mph or less and 32” for roadway design speed exceeding 45 mph.)

Existing railings – both in compliance and not in compliance – with MASH, NCHRP Report 350, and NCHRP Report 230 may be altered to improve the performance of the existing railings (i.e., raised to meet the minimum rail height requirement) where it is not feasible to install an approved railing. Coordination with BOS and BPD is required.

<u>NHS Structures</u>: Existing railing that does not comply with MASH, NCHRP Report 350, or NCHRP Report 230 and does not meet the minimum rail height requirement shall be upgraded to comply with MASH or NCHRP Report 350.

<u>Non-NHS Structures</u>: It is strongly encouraged that existing railing that does not comply with MASH, NCHRP Report 350, or NCHRP Report 230 and does not meet the minimum rail height requirement be upgraded to comply with MASH or NCHRP Report 350.</td></tr>
<tr><td>If rehabilitation work, as part of the 3R project, is scheduled or performed which widens the structure to either side, redecks (full-depth) any complete span of the structure, or if any work affecting the rail is done to the existing structure.</td><td>All railing on the structure must comply with MASH or NCHRP Report 350.

Limited project by project exceptions may be granted based on coordination and input by the Bureau of Structures and the Wisconsin Division of FHWA Structures Engineer.</td></tr>
<tr><td colspan="2">4R (Resurfacing, Restoration, Rehabilitation, Reconstruction)</td><td>All railing on the structure must comply with MASH or NCHRP Report 350.

Limited project by project exceptions may be granted based on coordination and input by the Bureau of Structures and the Wisconsin Division of FHWA Structures Engineer.</td></tr>
</table>

Table 30.8-1

WisDOT Requirements for Retrofitting/Upgrading Bridge Railings to Current Standards

✳ Examples of Preventative Maintenance projects include, but are not limited to:

1. Bridge deck work: Concrete deck repair, patching, and concrete overlays; asphaltic overlays; epoxy and polymer overlays; expansion joint replacement when done in

conjunction with an overlay or expansion joint elimination; chloride extraction; installation of a cathodic protection system.

2. Superstructure and substructure work: Steel structure cleaning and repainting, including complete repainting, zone painting, and spot painting with overcoat; structural repairs (except vehicle impact damage); bearing repair or replacement.

✳✳ Examples of 3R projects include, but are not limited to:

1. Bridge deck work: Bridge deck widenings and re-decks; expansion joint replacement when done in conjunction with an overlay or expansion joint elimination; approach slab replacement.

2. Superstructure and substructure work: Wing wall replacement; emergency bridge repair; structural repairs to railings based on vehicle impact damage;

The minimum railing height shall be measured from the top inside face of the railing to the top of the roadway surface at the toe of railing.

For all railing rehabilitations that require upgrades to comply with MASH or NCHRP Report 350, railings shall be employed as discussed in Section 30.2.

The following is a list of typical railing types that are in service on structures in Wisconsin. The underlined railings comply with MASH, NCHRP Report 350, or NCHRP Report 230 and may remain in service within rehabilitation projects. The *italicized* railings do not comply with MASH, NCHRP Report 350, or NCHRP Report 230 and shall be removed from service within rehabilitation projects.

1. Single slope parapet “32SS”, “36SS”, “42SS”, “56SS”. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 350 specifications and meet crash test criteria for TL-4.

2. Sloped face parapet "LF". Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

3. Sloped face parapet "HF". Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

4. Vertical face parapet “A”. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

5. Aesthetic parapet “TX”. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-2 based on a May 1997 FHWA memorandum.

6. Type “PF” tubular railing. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-2 based on a May 1997 FHWA memorandum.

7. Type “H” railing. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

8. Timber Railing. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-2 based on a May 1997 FHWA memorandum.

9. Type "W" railing. Railing may be used for rehabilitation projects on non-NHS structures only. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-3 based on a May 1997 FHWA memorandum.

10. Type “M” railing. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 350 and meets TL-4

11. *Type “F” railing.* Railing may not be used for rehabilitation projects.

12. Sloped face parapet “B”. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230.

The region shall contact the Bureau of Structures Development Section to determine the sufficiency of existing railings not listed above.

Rehabilitation or improvement projects to historically significant bridges require special attention. Typically, if the original railing is present on a historic bridge, it will likely not meet current crash testing requirements. In some cases, the original railing will not meet current minimum height and opening requirements. There are generally two different options for upgrading railings on historically significant bridges – install a crash-tested Traffic Railing to the interior side of the existing railing and leave the existing railing in place or replace the existing railing with a crash-tested Traffic Railing. Other alternatives may be available but consultation with the Bureau of Structures Development Section is required.

30.9 Railing Guidance for Railroad Structures

Per an April 2013 memorandum written by M. Myint Lwin, Director of the FHWA Office of Bridge Technology, bridge parapets, railings, and fencing shall conform to the following requirements when used in the design and construction of grade separated highway structures over railroads:

3. For NHS bridges over railroad:

   Bridge railings shall comply with AASHTO standards. For Federal-aid highway projects, the designer shall follow normal bridge railing specifications, design standards, and guidelines.

   However, railings for use on NHS bridges over railroads shall be governed by the railroad's standards, regardless of whether the bridge is owned by the railroad or WisDOT. For the case where an NHS bridge crosses over railroads operated by multiple authorities with conflicting parapet, railing, or fencing requirements, standards as agreed by the various railroad authorities and as approved by WisDOT shall be used.

4. For non-NHS bridges over railroad:

   Bridge railings shall comply with the policies outlined within this chapter. For Federal-aid highway projects, the designer shall follow normal bridge railing specifications, design standards, and guidelines.

   All federally funded non-NHS bridges including those over railroads shall be governed by WisDOT's policies outlined above, even if they differ from the railroad's standards.

30.10 References

1. American Association of State Highway and Transportation Officials. *AASHTO LRFD Bridge Design Specifications.*

2. American Association of State Highway and Transportation Officials. *Manual for Assessing Safety Hardware*.

3. National Cooperative Highway Research Program. *NCHRP Report 554 – Aesthetic Concrete Barrier Design*.

4. State of California, Department of Transportation. *Crash Testing of Various Textured Barriers*.

5. National Cooperative Highway Research Program. *NCHRP Report 350 – Recommended Procedures for the Safety Performance Evaluation of Highway Features*.

6. State of Wisconsin, Department of Transportation. *Facilities Development Manual*.

7. State of Wisconsin, Department of Transportation. *Wisconsin Bicycle Facility Design Handbook.*

8. State of Wisconsin, Department of Transportation. *Memorandum of Understanding between Wisconsin Department of Transportation, Wisconsin County Highway Association, and Transportation Builders Association.*

Table of Contents

36.1 Design Method

36.1.1 Design Requirements

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

36.1.2 Rating Requirements

The current version of *AASHTO Manual for Bridge Evaluation (LRFR)* covers rating of concrete box culverts. Currently, the *Bureau of Structures* does not require rating calculations for box culverts. See 45.8 for values to place on the plans for inventory and operating rating factors.

WisDOT Policy Item:

Current WisDOT policy is to not rate box culverts. In the future, rating requirements will be introduced as *AASHTO Manual for Bridge Evaluation (LRFR)* is updated to more thoroughly address box culverts.

36.1.3 Standard Permit Design Check

New structures are also to be checked for strength for the 190 kip Wisconsin Standard Permit Vehicle (Wis-SPV), with a single lane loaded, multiple presence factor equal to 1.0, and a live load factor (γ_{LL}) as shown in Table 45.3-3. See section 45.6 for the configuration of the Wis-SPV. The structure should have a minimum capacity to carry a gross vehicle load of 190 kips, while also supporting the future wearing surface (where applicable – future wearing surface loads are only applied to box culverts with no fill). When applicable, this truck will be designated as a Single Trip Permit Vehicle. It will have no escorts restricting the presence of other traffic on the culvert, no lane position restrictions imposed and no restrictions on speed to reduce the dynamic load allowance, IM. The maximum Wisconsin Standard Permit Vehicle load that the structure can resist, calculated including current wearing surface loads, is shown on the plans. The current version of *AASHTO Manual for Bridge Evaluation (LRFR)* does not thoroughly cover rating of concrete box culverts. See 45.8 for values to place on the plans for maximum (Wis-SPV) vehicle load.

36.14 Design Example

E36-1 Twin Cell Box Culvert LRFD

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Top Slab Unfactored Dead Load Moments (kip-ft)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	-0.04	-1.14	-5.47	-1.18	0.00
0.1	0.73	1.45	-4.67	-1.00	0.00
0.2	1.27	3.32	-3.87	-0.83	0.00
0.3	1.60	4.48	-3.07	-0.66	0.00
0.4	1.69	4.93	-2.27	-0.49	0.00
0.5	1.56	4.67	-1.47	-0.32	0.00
0.6	1.21	3.69	-0.67	-0.15	0.00
0.7	0.63	2.01	0.13	0.03	0.00
0.8	-0.18	-0.39	0.93	0.20	0.00
0.9	-1.21	-3.50	1.72	0.37	0.00
1.0	-2.46	-7.32	2.52	0.54	0.00

Bottom Slab Unfactored Dead Load Moments (kip-ft)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	-0.60	-0.17	-7.63	-1.42	0.00
0.1	1.36	2.26	-6.51	-1.21	0.00
0.2	2.76	3.98	-5.39	-1.00	0.00
0.3	3.61	4.99	-4.27	-0.79	0.00
0.4	3.91	5.29	-3.15	-0.59	0.00
0.5	3.65	4.87	-2.03	-0.38	0.00
0.6	2.85	3.75	-0.90	-0.17	0.00
0.7	1.49	1.91	0.22	0.04	0.00
0.8	-0.42	-0.64	1.34	0.25	0.00
0.9	-2.88	-3.90	2.46	0.46	0.00
1.0	-5.89	-7.88	3.58	0.67	0.00

Exterior Wall Unfactored Dead Load Shears (kip)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	0.09	-0.08	4.78	0.73	0.00
0.1	0.09	-0.08	3.60	0.59	0.00
0.2	0.09	-0.08	2.50	0.45	0.00
0.3	0.09	-0.08	1.49	0.30	0.00
0.4	0.09	-0.08	0.56	0.16	0.00
0.5	0.09	-0.08	-0.27	0.01	0.00
0.6	0.09	-0.08	-1.03	-0.13	0.00
0.7	0.09	-0.08	-1.69	-0.27	0.00
0.8	0.09	-0.08	-2.27	-0.42	0.00
0.9	0.09	-0.08	-2.76	-0.56	0.00
1.0	0.09	-0.08	-3.17	-0.71	0.00

Interior Wall Unfactored Dead Load Shears (kip)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	0.00	0.00	0.00	0.00	0.00
0.1	0.00	0.00	0.00	0.00	0.00
0.2	0.00	0.00	0.00	0.00	0.00
0.3	0.00	0.00	0.00	0.00	0.00
0.4	0.00	0.00	0.00	0.00	0.00
0.5	0.00	0.00	0.00	0.00	0.00
0.6	0.00	0.00	0.00	0.00	0.00
0.7	0.00	0.00	0.00	0.00	0.00
0.8	0.00	0.00	0.00	0.00	0.00
0.9	0.00	0.00	0.00	0.00	0.00
1.0	0.00	0.00	0.00	0.00	0.00

Calculate the nominal moment capacity of the rectangular section in accordance with **LRFD [5.7.3.2.3]**:

$a := \beta_1 \cdot c$ $\boxed{a = 0.83}$ in

$M_n := \left[A_s \cdot f_s \cdot \left(d_s - \frac{a}{2}\right) \frac{1}{12}\right]$ $\boxed{M_n = 23.0}$ kip-ft

For reinforced concrete cast-in-place box structures, $\phi_f = 0.90$ **LRFD [Table 12.5.5-1]**. Therefore the usable capacity is:

$M_r := \phi_f \cdot M_n$ $\boxed{M_r = 20.7}$ kip-ft

The required capacity:

Corner Moment $\boxed{Mstr1_{CB} = 17.3}$ kip-ft

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

$b = 12.0$ in   width of the concrete design section, in

$h = 12.0$ in   height of the concrete design section, in

$f_r := 0.24 \cdot \sqrt{f'_c}$ modulus of rupture, ksi **LRFD [5.4.2.6]** $f_r = 0.45$ ksi

$I_g := \frac{1}{12} \cdot b \cdot h^3$ gross moment of inertia, in⁴ $I_g = 1728.00$ in^4

$\frac{h}{2} = 6.0$ distance from the neutral axis to the extreme element

$S_c := \frac{I_g}{\frac{h}{2}}$ section modulus, in^3 $S_c = 288.00$ in^3

The corresponding cracking moment is:

$M_{cr} = \gamma_3(\gamma_1 \cdot f_r)S_c$ therefore, $M_{cr} = 1.1(f_r)S_c$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement for A615, Grade 60 reinforcement

$M_{cr} := 1.1 f_r \cdot S_c \cdot \frac{1}{12}$ $\boxed{M_{cr} = 11.9}$ kip-ft

$\boxed{1.33 \cdot Mstr1_{CB} = 23.1}$ kip-ft

Is $M_r = 20.7$ kip-ft greater than the lesser of M_{cr} and 1.33*M_{str}? $\boxed{\text{check} = \text{"OK"}}$

Per **LRFD [5.7.3.4]**, the spacing(s) of reinforcement in the layer closest to the tension face shall satisfy:

$$s \le \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c \qquad \text{in which:} \qquad \beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)}$$

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

$h = 12.0$ height of the concrete design section, in

Calculate the ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face:

$$\beta_s := 1 + \frac{d_c}{0.7 \cdot (h - d_c)} \qquad \boxed{\beta_s = 1.34}$$

Calculate the reinforcement ratio:

$$\rho := \frac{A_s}{b \cdot d_s} \qquad \boxed{\rho = 0.0043}$$

Calculate the modular ratio:

$$N := \frac{E_s}{E_c} \qquad \boxed{N = 8.06}$$

Calculate f_{ss}, the tensile stress in steel reinforcement at the Service I Limit State (ksi). The moment arm used in the equation below to calculate f_{ss} is: (j) (h-d_c)

$$k := \sqrt{(\rho \cdot N)^2 + (2 \cdot \rho \cdot N)} - \rho \cdot N \qquad \boxed{k = 0.2301}$$

$$j := 1 - \frac{k}{3} \qquad \boxed{j = 0.9233}$$

$Ms1_{CB} = 11.18$ service moment, kip-ft

$$f_{ss} := \frac{Ms1_{CB} \cdot 12}{A_s \cdot (j) \cdot (h - d_c)} \le 0.6\ f_y \qquad \boxed{f_{ss} = 30.23} \text{ ksi} \quad \le 0.6\ f_y \text{ O.K.}$$

Calculate the maximum spacing requirements per **LRFD [5.10.3.2]**:

Is $M_{r2} = 10.6$ kip-ft greater than the lesser of M_{cr} and 1.33*M_{str}? $\boxed{check = \text{"OK"}}$

$\boxed{M_{cr} = 11.9}$ kip-ft

$\boxed{1.33 \cdot Mstr1_{CBV2} = 10.5}$ kip-ft

Calculate f_{ss}, the tensile stress in steel reinforcement at the Service I Limit State (ksi).

$Ms1_{CBV2} = 3.43$ service moment at the second cutoff location, kip-ft

$$f_{ss2} := \frac{Ms1_{CBV2} \cdot 12}{A_s \cdot (j) \cdot (h - d_c)}$$ $\boxed{f_{ss} = 30.23}$ ksi

Calculate the maximum spacing requirements per **LRFD [5.10.3.2]**:

$$s_{max2\_1} := \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss2}} - 2 \cdot d_c$$ $s_{max2\_1} = 51.69$ in

$s_{max2\_2} := s_{max2}$ $s_{max2\_2} = 18.00$ in

$s_{max} := \min(s_{max2\_1}, s_{max2\_2})$ $\boxed{s_{max} = 18.00}$ in

Check that the provided spacing (for half of the bars) is less than the maximum allowable spacing

$spacing2 := 2 \cdot spacing$ $spacing2 = 15.00$ in

Is $spacing2 = 15.00$ in $\leq$ $s_{max} = 18.00$ in $\boxed{check = \text{"OK"}}$

Extension Lengths:

The extension lengths for the corner bars are shown below. Calculations for other bars are similar.

Extension lengths for general reinforcement per **LRFD [5.11.1.2.1]**:

$$MaxDepth := \max\left(t_{ts} - cover, t_{wex} - cover, t_{bs} - cover_{bot}\right) \qquad MaxDepth = 11.00 \quad in$$

Effective member depth $\dfrac{MaxDepth - \frac{1}{2} Bar_D\left(Bar_{No\_CB}\right)}{12} = 0.89$ ft

15 x bar diameter $\dfrac{15 \cdot Bar_D\left(Bar_{No\_CB}\right)}{12} = 0.78$ ft

1/20 times clear span $\dfrac{\max\left(W_1, W_2\right)}{20} = 0.60$ ft

The maximum of the values listed above:

$$ExtendLength\_gen_{CB} = 0.89 \quad ft$$

Extension lengths for negative moment reinforcement per **LRFD [5.11.1.2.3]**:

Effective member depth $\dfrac{MaxDepth - \frac{1}{2} Bar_D\left(Bar_{No\_CB}\right)}{12} = 0.89$ ft

12 x bar diameter $\dfrac{12 \cdot Bar_D\left(Bar_{No\_CB}\right)}{12} = 0.63$ ft

0.0625 times clear span $0.0625 \max\left(W_1, W_2\right) = 0.75$ ft

The maximum of the values listed above:

$$ExtendLength\_neg_{CB} = 0.89 \quad ft$$

The development length:

$$DevLength_{CB} = 1.00 \quad ft$$

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40.8 Widenings

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

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

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

For multi-columned piers, consider the cap connection to the existing cap as pinned and design the new portion to the latest LRFD criteria. The new column(s) are not required to meet AASHTO 3.6.5 (600 kip loading) as a widening is considered rehabilitation. *BOS intends to provide standard details in the Bridge Manual for a crash barrier that could, at the option of the Region, be used to strengthen and provide motorists protection for existing piers, including widenings.*

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

For foundation support, use the most current method available.

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

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

40.9 Superstructure Replacements/Moved Girders (with Widening)

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

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

Evaluate the existing piers using current LRFD criteria. If an existing multi-columned pier has 3 or more columns, the 600 kip vehicular impact loading need not be considered if the pier is adjacent to a roadway with a design speed ≤ 40 mph. If the design speed is 45 mph or 50 mph, the 600 kip vehicular impact loading need not be considered if a minimum of "vehicle protection" is provided as per FDM 11-35-1. For design speeds > 50 mph, all criteria as per 13.4.10 must be met.

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

The superstructure shall be designed to current LRFD criteria.

Anchor Size, d_a	"Type S" Anchors Mechanical Min. h_{ef} (in)	"Type S" or "Type L" Anchors Adhesive (Dry Concrete) Min. Bond Stress, τ_{uncr} (psi)	 Adhesive (Dry Concrete) Min. Bond Stress, τ_{cr} (psi)	 Adhesive (Water-Saturated Concrete) Min. Bond Stress, τ_{uncr} (psi)	 Adhesive (Water-Saturated Concrete) Min. Bond Stress, τ_{cr} (psi)
#4 or 1/2"	2.5	1184	552	510	386
#5 or 5/8"	2.5	1184	563	722	476
#6 or 3/4"	3.0	1184	608	709	506
#7 or 7/8"	3.5	1184	608	682	434
#8 or 1"	4.0	1184	608	807	531
#9 or 1-1/8"	4.5	1025	601	681	465

Table 40.16-1

Tension Design Table for Concrete Masonry Anchors, "Type S" and "Type L"

The minimum bond stress values for adhesive anchors in Table 40.16-1 are based on the Approved Products List for "Concrete Masonry Anchors, Type L". The designer shall determine whether the concrete masonry adhesive anchors are to be utilized in dry concrete (i.e., rehabilitation locations where concrete is fully cured, etc.) or water-saturated concrete (i.e., new bridge decks, box culverts, etc.) and shall design the anchors accordingly.

The factored tension force on each anchor, N_u, must be less than or equal to the factored tensile resistance, N_r. For mechanical anchors:

$$N_r = \phi_{ts}N_{sa} \le \phi_{tc}N_{cb} \le \phi_{tc}N_{pn}$$

In which:

ϕ_{ts} = Strength reduction factor for anchors in concrete, **ACI [D.4.3]**
= 0.65 for brittle steel as defined in 40.16.2
= 0.75 for ductile steel as defined in 40.16.2

N_{sa} = Nominal steel strength of anchor in tension, **ACI [D.5.1.2]**
= $A_{se,N}f_{uta}$

$A_{se,N}$ = Effective cross-sectional area of anchor in tension (in^2)

f_{uta} = Specified tensile strength of anchor steel (psi)
≤ $1.9f_{ya}$
≤ 125 ksi

f_{ya} = Specified yield strength of anchor steel (psi)

ϕ_{tc} = Strength reduction factor for anchors in concrete
= 0.65 for anchors without supplementary reinforcement per 40.16.3
= 0.75 for anchors with supplementary reinforcement per 40.16.3

N_{cb} = Nominal concrete breakout strength in tension, **ACI [D.5.2.1]**

$$= \frac{A_{Nc}}{9(h_{ef})^2}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$$

A_{Nc} = Projected concrete failure area of a single anchor, see Figure 40.16-1

$$= (S_1 + S_2)(S_3 + S_4)$$

h_{ef} = Effective embedment depth of anchor per Table 40.16-1. May be reduced per **ACI [D.5.2.3]** when anchor is located near three or more edges.

$\psi_{ed,N}$ = Modification factor for tensile strength based on proximity to edges of concrete member, **ACI [D.5.2.5]**
= 1.0 if $c_{a,min} \geq 1.5h_{ef}$

$$= 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}} \text{ if } c_{a,min} < 1.5h_{ef}$$

$c_{a,min}$ = Minimum edge distance from center of anchor shaft to the edge of concrete, see Figure 40.16-1 (in)

$\psi_{c,N}$ = Modification factor for tensile strength of anchors based on the presence or absence of cracks in concrete, **ACI [D.5.2.6]**
= 1.0 when post-installed anchors are located in a region of a concrete member where analysis indicates cracking at service load levels
= 1.4 when post-installed anchors are located in a region of a concrete member where analysis indicates no cracking at service load levels

$\psi_{cp,N}$ = Modification factor for post-installed anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation, **ACI [D.5.2.7]**
= 1.0 if $c_{a,min} \geq c_{ac}$

$$= \frac{c_{a,min}}{c_{ac}} \geq \frac{1.5h_{ef}}{c_{ac}} \text{ if } c_{a,min} < c_{ac}$$

E45-2.3 Composite Girder Section Properties

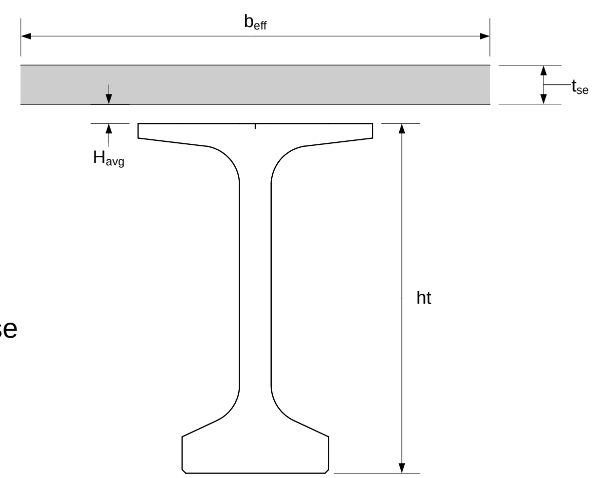
Calculate the effective flange width in accordance with 17.2.11 and **LRFD [4.6.2.6]**:

$b_{eff} := S \cdot 12$ $\boxed{b_{eff} = 90.00}$ in

The effective width, b_{eff}, must be adjusted by the modular ratio, n, to convert to the same concrete material (modulus) as the girder.

$b_{eadj} := \frac{b_{eff}}{n}$ $\boxed{b_{eadj} = 58.46}$ in

Calculate the composite girder section properties:



effective slab thickness; $\boxed{t_{se} = 7.50}$ in

effective slab width; $\boxed{b_{eadj} = 58.46}$ in

haunch thickness; $\boxed{H_{avg} = 2.00}$ in

total height; $h_c := ht + H_{avg} + t_{se}$

$\boxed{h_c = 81.50}$ in

$\boxed{n = 1.540}$

Note: The area of the concrete haunch is not included in the calculation of the composite section properties.

Component	Ycg	A	AY	AY^2	I	I+AY^2
Deck	77.75	438	34089	2650458	2055	2652513
Girder	34.87	915	31906	1112564	656426	1768990
Haunch	73	0	0	0	0	0
Summation		1353	65996			4421503

$\Sigma A := 1353$ in^2

$\Sigma AY := 65996$ in^3

$\Sigma IplusAYsq := 4421503$ in^4

$y_{cgb} := \frac{-\Sigma AY}{\Sigma A}$ $\boxed{y_{cgb} = -48.8}$ in

$y_{cgt} := ht + y_{cgb}$ $\boxed{y_{cgt} = 23.2}$ in

$A_{cg} := \Sigma A$ $\boxed{A_{cg} = 1353}$ in^2

$I_{cg} := \Sigma IplusAYsq - A_{cg} \cdot y_{cgb}^{\ 2}$ $\boxed{I_{cg} = 1202381}$ in^4

$S_{cgt} := \frac{I_{cg}}{y_{cgt}}$ $\boxed{S_{cgt} = 51777}$ in^3

$S_{cgb} := \frac{I_{cg}}{y_{cgb}}$ $\boxed{S_{cgb} = -24650}$ in^3

E45-2.4 Dead Load Analysis - Interior Girder

Dead load on non-composite (DC_1):

weight of 72W girders $\boxed{w_g = 0.953}$ klf

weight of 2-in haunch

$w_h := \left(\frac{H_{avg}}{12}\right) \cdot \left(\frac{b_{tf}}{12}\right) \cdot (w_c)$ $\boxed{w_h = 0.100}$ klf

weight of diaphragms $\boxed{w_D := 0.006}$ klf

weight of slab

$w_d := \left(\frac{t_s}{12}\right) \cdot (S) \cdot (w_c)$ $\boxed{w_d = 0.750}$ ksf

$DC_1 := w_g + w_h + w_D + w_d$ $\boxed{DC_1 = 1.809}$ klf

$V_{DC1} := \frac{DC_1 \cdot L}{2}$ $\boxed{V_{DC1} = 132}$ kips

$M_{DC1} := \frac{DC_1 \cdot L^2}{8}$ $\boxed{M_{DC1} = 4820}$ kip-ft