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2.1 Organizational Charts

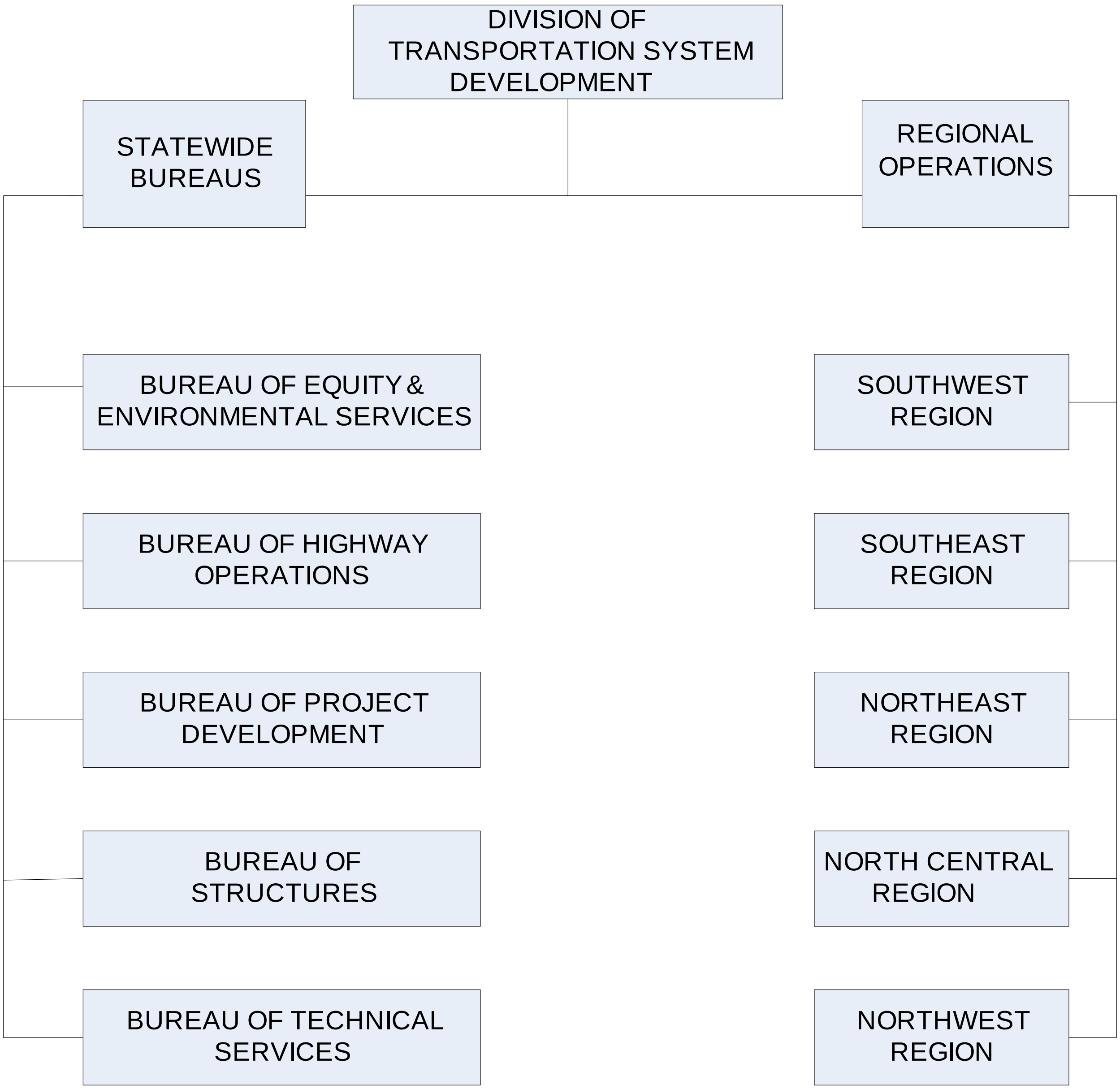


Figure 2.1-1
Division of Transportation System Development

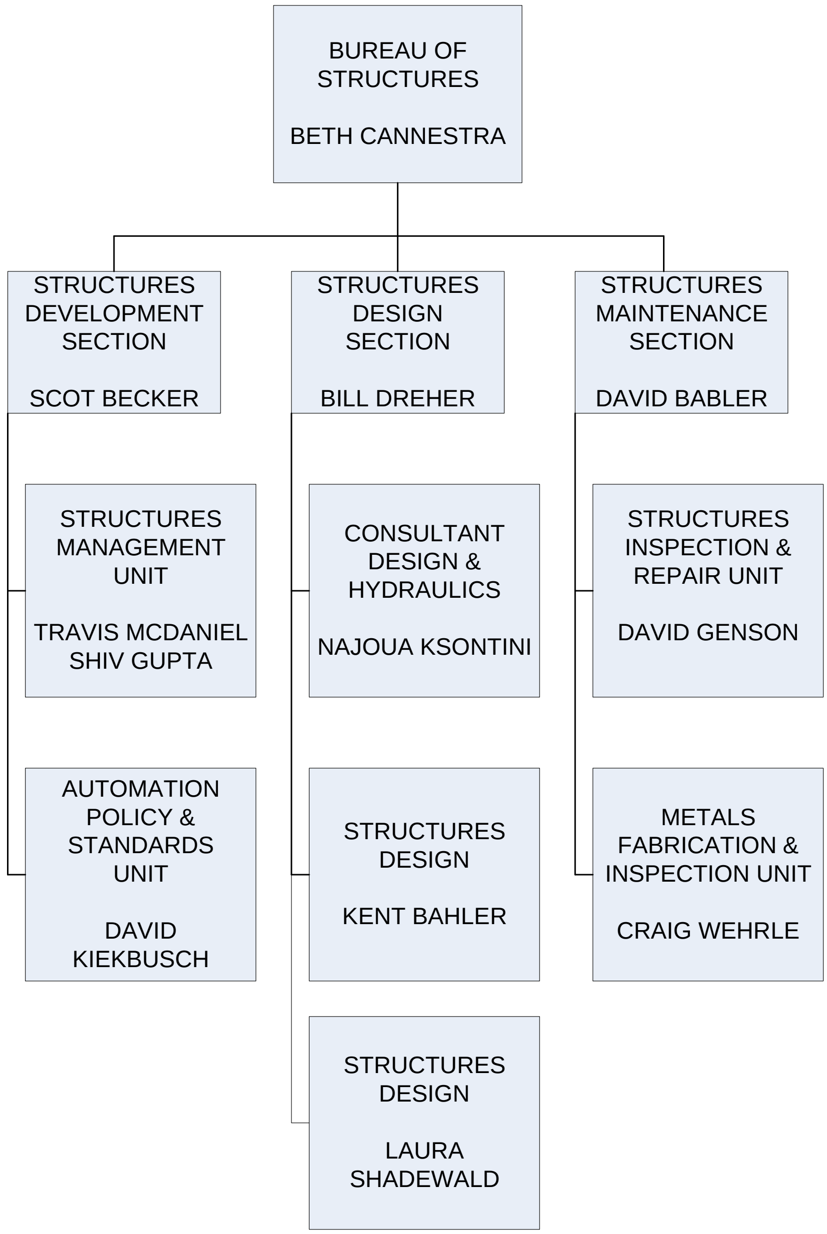
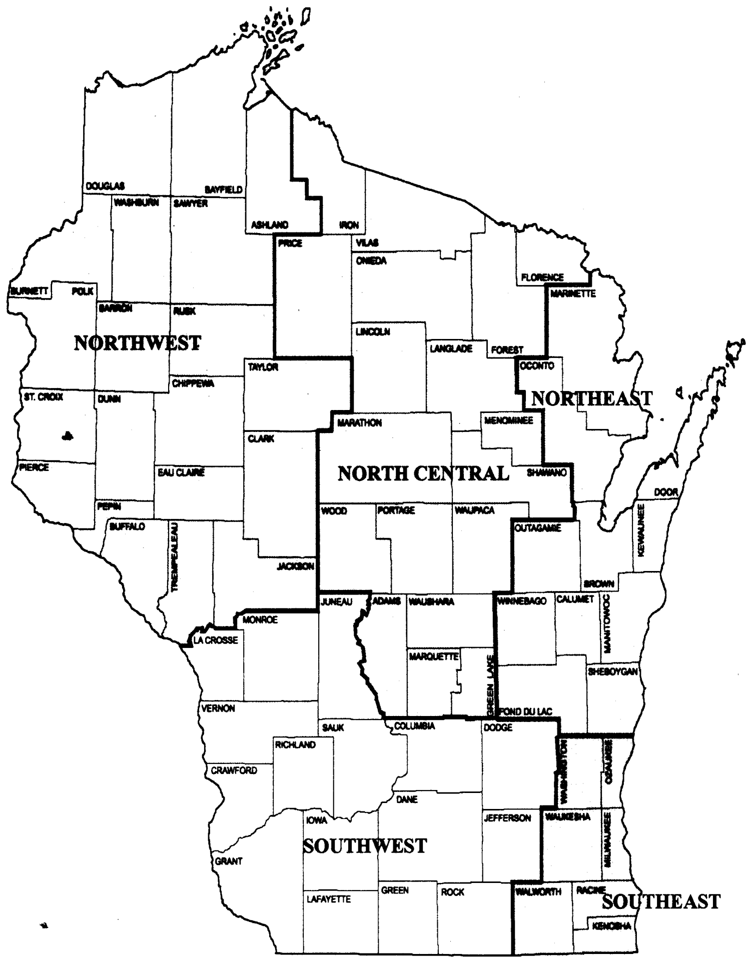


Figure 2.1-2
Bureau of Structures



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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5.1 Factors Governing Bridge Costs

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

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

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

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

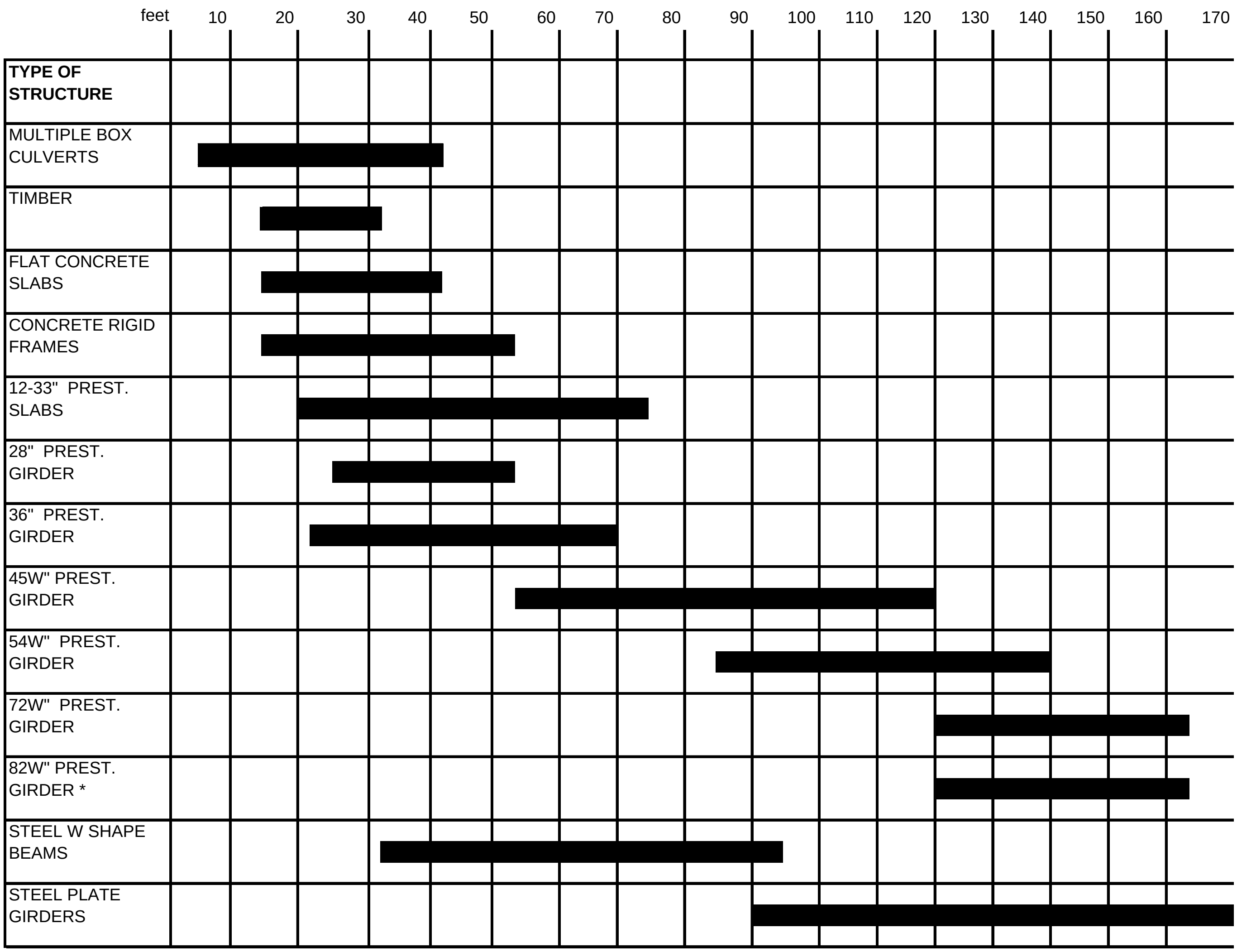
In these reports prestressed girder costs are grouped together because there is a small cost difference between girder sizes. Refer to unit costs. Concrete slab costs are also grouped together for this reason.

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

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

Note: Current costs are given in English units.

5.2 Economic Span Lengths



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

Figure 5.2-1
Economic Span Lengths

5.3 Contract Unit Bid Prices

Item No.	Bid Item	Unit	Cost
206.6010.S	Temporary Shoring	LS	--
210.0100	Backfill Structure	CY	14.13
303.0115	Pit Run	CY	9.63
311.0115	Breaker Run	CY	16.95
502.0100	Concrete Masonry Bridges	CY	418.25
502.1100	Concrete Masonry Seal	CY	144.70
502.2000	Compression Joint Sealer Preformed Elastomeric (width)	LF	45.00
502.3100	Expansion Device (structure) (LS)	LF	118.25
502.3110.S	Expansion Device Modular (structure) (LS)	LF	177.00
502.3200	Protective Surface Treatment	SY	2.34
502.6500	Protective Coating Clear	GAL	57.50
503.0128	Prestressed Girder Type I 28-Inch	LF	123.50
503.0136	Prestressed Girder Type I 36-Inch	LF	124.43
503.0137	Prestressed Girders Type I 36W-Inch	LF	166.00
503.0145	Prestressed Girder Type I 45-Inch	LF	139.00
503.0146	Prestressed Girders Type I 45W-Inch	LF	149.63
503.0154	Prestressed Girder Type I 54-Inch	LF	184.00
503.0155	Prestressed Girder Type I 54W-Inch	LF	157.25
503.0170	Prestressed Girder Type I 70-Inch	LF	163.79
503.0172	Prestressed Girders Type I 72W-Inch	LF	178.19
503.0182	Prestressed Girder Type I 82W-Inch	LF	185.00
504.0100	Concrete Masonry Culverts	CY	441.10
504.0500	Concrete Masonry Retaining Walls	CY	366.75
505.0405	Bar Steel Reinforcement HS Bridges	LB	0.70
506.2605	Bar Steel Reinforcement HS Culverts	LB	0.77
506.2610	Bar Steel Reinforcement HS Retaining Walls	LB	0.57
506.3005	Bar Steel Reinforcement HS Coated Bridges	LB	0.69
506.3010	Bar Steel Reinforcement HS Coated Culverts	LB	0.77
506.3015	Bar Steel Reinforcement HS Coated Retaining Walls	LB	0.67
506.0105	Structural Carbon Steel	LB	1.62
506.0605	Structural Steel HS	LB	1.42
506.2605	Bearing Pads Elastomeric Non-Laminated	EACH	74.95
506.2610	Bearing Pads Elastomeric Laminated	EACH	1,075.00
506.3005	Welded Shear Stud Connectors 7/8 x 4-Inch	EACH	2.77
506.3010	Welded Shear Stud Connectors 7/8 x 5-Inch	EACH	5.42
506.3015	Welded Shear Stud Connectors 7/8 x 6-Inch	EACH	3.32
506.3020	Welded Shear Stud Connectors 7/8 x 7-Inch	EACH	3.18
506.3025	Welded Shear Stud Connectors 7/8 x 8-Inch	EACH	3.10
506.4000	Steel Diaphragms (structure)	EACH	511.45
506.5000	Bearing Assemblies Fixed (structure)	EACH	1,308.00
506.6000	Bearing Assemblies Expansion (structure)	EACH	1190.00
507.0200	Treated Lumber and Timber	MBM	6225.00
508.1600	Piling Treated Timber Delivered	LF	20.00
510.2005	Preboring Cast-in-Place Concrete Piling	LF	75.00
510.3021	Piling CIP Concrete Delivered and Driven 10 ¾ x 0.219-Inch	LF	31.30

510.3030	Piling CIP Concrete Delivered and Driven 10 ¾ x 0.25-Inch	LF	30.89
510.3040	Piling CIP Concrete Delivered and Driven 10 ¾ x 0.365-Inch	LF	33.40
510.3050	Piling CIP Concrete Delivered and Driven 10 ¾ x 0.5-Inch	LF	--
510.3023	Piling CIP Concrete Delivered and Driven 12 ¾ x 0.219-Inch	LF	29.00
510.3033	Piling CIP Concrete Delivered and Driven 12 ¾ x 0.25-Inch	LF	32.91
510.3043	Piling CIP Concrete Delivered and Driven 12 ¾ x 0.375-Inch	LF	39.55
510.3053	Piling CIP Concrete Delivered and Driven 12 ¾ x 0.5-Inch	LF	50.00
510.3024	Piling CIP Concrete Delivered and Driven 14 x 0.219-Inch	LF	--
510.3034	Piling CIP Concrete Delivered and Driven 14 x 0.25-Inch	LF	33.94
510.3044	Piling CIP Concrete Delivered and Driven 14 x 0.375-Inch	LF	43.85
510.3054	Piling CIP Concrete Delivered and Driven 14 x 0.5-Inch	LF	35.00
510.3026	Piling CIP Concrete Delivered and Driven 16 x 0.219-Inch	LF	--
510.3036	Piling CIP Concrete Delivered and Driven 16 x 0.25-Inch	LF	--
510.3046	Piling CIP Concrete Delivered and Driven 16 x 0.375-Inch	LF	--
510.3056	Piling CIP Concrete Delivered and Driven 16 x 0.5-Inch	LF	56.60
511.2105	Piling Steel Delivered and Driven HP 10-Inch x 42 LB	LF	28.45
511.2110	Piling Steel Delivered and Driven HP 12-Inch x 53 LB	LF	32.34
511.2115	Piling Steel Delivered and Driven HP 12-Inch x 74 LB	LF	38.77
511.2120	Piling Steel Delivered and Driven HP 14-Inch x 73 LB	LF	36.90
511.3000	Pile Points	EACH	90.82
511.6000	Piling Steel Preboring	LF	151.50
512.1000	Piling Steel Sheet Temporary	SF	20.00
513.4050	Railing Tubular Type F (structure) (LS)	LF	95.00
513.4052 or 3	Railing Tubular Type F- (4 or 5) Modified (structure) (LS)	LF	133.00
513.4055	Railing Tubular Type H (structure) (LS)	LF	98.10
513.4060	Railing Tubular Type M (structure) (LS)	LF	148.60
513.4065	Railing Tubular Type PF (structure) (LS)	LF	--
513.4090	Railing Tubular Screening Structure B-	LF	102.20
513.7050	Railing Type W Structure B-	LF	126.50
00000	Concrete Railing, "Texas Rail"	LF	158.75
00000	Concrete Parapet, Type 'LF' & 'A' (estimate)	LF	80.00
513.7005	Railing Steel Type C1 (structure) (LS)	LF	153.50
513.7010	Railing Steel Type C2 (structure) (LS)	LF	113.30
513.7015	Railing Steel Type C3 (structure) (LS)	LF	79.10
513.7020	Railing Steel Type C4 (structure) (LS)	LF	--
513.7025	Railing Steel Type C5 (structure) (LS)	LF	102.70
513.7030	Railing Steel Type C6 (structure) (LS)	LF	80.00
514.0445	Floor Drains Type GC	EACH	1695.00
514.2625	Downspouts 6-Inch	LF	--

516.0500	Rubberized Membrane Waterproofing	SY	25.42
517.1010.S	Concrete Staining	SF	1.25
517.1015.S	Concrete Staining Multi-Color	SF	2.10
517.1050.S	Architectural Surface Treatment	SF	3.50
604.0400	Slope Paving Concrete	SY	53.00
604.0500	Slope Paving Crushed Aggregate	SY	17.00
604.0600	Slope Paving Select Crushed Material	SY	17.00
606.0100	Riprap Light	CY	45.00
606.0200	Riprap Medium	CY	51.00
606.0300	Riprap Heavy	CY	45.00
606.0500	Grouted Riprap Light	CY	60.00
606.0600	Grouted Riprap Medium	CY	101.96
606.0700	Grouted Riprap Heavy	CY	58.60
612.0106	Pipe Underdrain 6-Inch	LF	5.37
612.0206	Pipe Underdrain Unperforated 6-Inch	LF	8.75
612.0406	Pipe Underdrain Wrapped 6-Inch	LF	7.25
616.0205	Fence Chain Link 5-FT	LF	39.12
616.0206	Fence Chain Link 6-FT	LF	47.92
616.0208	Fence Chain Link 8-FT	LF	--
645.0105	Geotextile Fabric Type C	SY	2.67
645.0111	Geotextile Fabric Type DF Schedule A	SY	2.00
645.0120	Geotextile Fabric Type HR	SY	3.41
652.0125	Conduit Rigid Metallic 2-Inch	LF	11.80
652.0135	Conduit Rigid Metallic 3-Inch	LF	24.15
652.0225	Conduit Rigid Nonmetallic Schedule 40 2-Inch	LF	4.00
652.0235	Conduit Rigid Nonmetallic Schedule 40 3-Inch	LF	6.50
SPV.0035	HPC Masonry Superstructure	CY	399.50
SPV.0165	Anti-Graffiti Coating	SF	1.83
SPV.0180	Pigmented Protective Surface Treatment	SY	5.70

Table 5.3-1

Contract Unit Bid Prices for New Structures

Item No.	Bid Item	Unit	Cost
455.0___	Asphaltic Material ________	TON	689.00
460.1___	HMA Pavement Type ____	TON	23.60
502.5002	Masonry Anchors Type L No. 4 Bars	EACH	11.07
502.5005	Masonry Anchors Type L No. 5 Bars	EACH	11.00
502.5010	Masonry Anchors Type L No. 6 Bars	EACH	21.00
502.5015	Masonry Anchors Type L No. 7 Bars	EACH	12.50
502.5020	Masonry Anchors Type L No. 8 Bars	EACH	17.00
502.5025	Masonry Anchors Type L No. 9 Bars	EACH	20.50
502.6102	Masonry Anchors Type S ½-Inch	EACH	21.85
502.6105	Masonry Anchors Type S 5/8-Inch	EACH	13.50
502.6110	Masonry Anchors Type S ¾-Inch	EACH	13.75
502.6115	Masonry Anchors Type S 7/8-Inch	EACH	--
502.6120	Masonry Anchors Type S 1-Inch	EACH	12.00
505.0904	Bar Couplers No. 4	EACH	11.56
505.0905	Bar Couplers No. 5	EACH	16.22
505.0906	Bar Couplers No. 6	EACH	27.25
505.0907	Bar Couplers No. 7	EACH	10.33
505.0908	Bar Couplers No. 8	EACH	46.64
505.0909	Bar Couplers No. 9	EACH	34.10
509.0301	Preparation Decks Type 1	SY	74.50
509.0302	Preparation Decks Type 2	SY	81.50
509.0500	Cleaning Decks	SY	9.00
509.1000	Joint Repair	SY	564.00
509.1200	Curb Repair	LF	41.40
509.1500	Concrete Surface Repair	SF	64.50
509.2000	Full-Depth Deck Repair	SY	270.00
509.2500	Concrete Masonry Overlay Decks	CY	425.20
SPV.0165	Epoxy Overlay	SF	12.00

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 2006 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	24	453,000	35,249,963	51.45	77.81
Reinforced Concrete Slabs (All But A5)	39	63,984	5,211,526	47.76	81.50
Reinf. Conc. Slab (A5 Abuts)	37	66,675	5,174,920	40.95	77.61
Prestressed Box Girders	4	12,761	1,804,149	62.28	141.38

Table 5.4-1
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	24	167,899	13,544,510	60.12	80.67
Steel Plate Girders	2	28,782	6,572,494	142.68	228.35
Reinf. Conc. Slabs (All But A5)	6	20,316	1,732,386	43.02	85.27
Reinf. Conc. Slabs (A5 Abuts)	4	5,979	605,133	44.08	101.21

Table 5.4-2
Grade Separation Structures

Box Culverts	No. of Culverts	Cost per Lin. Ft.
Single Cell	9	885.72
Twin Cell	6	1,805.23
Triple Cell	0	0
Aluminum	0	0

Table 5.4-3
Box Culverts

Bascule Bridge	Cost per Sq. Ft.
B-15-23	388.85

Table 5.4-4
Bascule Bridge

Retaining Walls	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Walls	7	6,102	343,147	56.25
Modular Walls	0	0	0	0
Concrete Walls	7	15,848	1,059,081	66.83
Panel Walls	16	25,907	2,908,492	112.07

Table 5.4-5
Retaining Walls

5.4.2 2007 Year End Structure Costs

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

Table 5.4-6
Stream Crossing Structures

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

Table 5.4-7
Grade Separation Structures

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

Table 5.4-8
Box Culverts

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

Table 5.4-9
Pedestrian Bridges

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

Table 5.4-10
Retaining Walls

5.4.3 2008 Year End Structure Costs

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

Table 5.4-11
Stream Crossing Structures

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

Table 5.4-12
Grade Separation Structures

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

Table 5.4-13
Box Culverts

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

Table 5.4-14
Pedestrian Bridges

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

Table 5.4-15
Railroad Bridges

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

Table 5.4-16
Bascule Bridges

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

Table 5.4-17
Retaining Walls

5.4.4 2009 Year End Structure Costs

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

Table 5.4-18
Stream Crossing Structures

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

Table 5.4-19
Grade Separation Structures

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

Table 5.4-20
Box Culverts

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

Table 5.4-21
Pedestrian Bridges

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

Table 5.4-22
County Timber Bridges

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

Table 5.4-23
Bascule Bridges

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

Table 5.4-24
Retaining Walls

5.4.5 2010 Year End Structure Costs

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

Table 5.4-25
Stream Crossing Structures

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

Table 5.4-26
Grade Separation Structures

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

Table 5.4-27
Box Culverts

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

Table 5.4-28
Pre-Fab Pedestrian Bridge

Pedestrian Bridges	Cost per Sq. Ft.
4	179.56

Table 5.4-29
Pedestrian Bridges

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

Table 5.4-30
Bascule Bridges

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

Table 5.4-31
Retaining Walls

Table of Contents

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6.1 Approvals, Distribution and Work Flow

Production of Structural Plans

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

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

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

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

6.2 Preliminary Plans

6.2.1 Structure Survey Report

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

Under the “Plans and Projects” heading:

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

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

When preparing the Structure Survey Report, designers will make their best estimate of structure type and location of substructure units. The completed Structure Survey Report with the locations of the substructure units and all required attachments and supporting information will then be submitted to the Geotechnical Section, attention Chief Geotechnical Engineer, through the Regional Soils Engineer, and to the Consultant Design & Hydraulics Unit, attention Consultant Design & Hydraulics Supervisor. This submittal will take place a minimum of 15 months in advance of the final plans due date shown on the Structure Survey Report. Under this plan, the box on the Structure Survey Report titled, "Have soil borings been requested" should always be checked "yes". The Geotechnical Section has responsibility for conducting the necessary soil borings. The Consultant Design and Hydraulics Unit and the Geotechnical Section will coordinate activities to deliver the completed preliminary plans on schedule.

In most instances, the geotechnical work will proceed after the receipt of the Structure Survey Report, but in advance of the development of the preliminary bridge plans. However, special circumstances may require that the preliminary bridge plans precede the geotechnical work. The Geotechnical Section may request preliminary bridge plans under the following conditions.

1. A review of available subsurface information indicates the probability of very shallow and highly variable bedrock.

2. The span on the Structure Survey Report falls in the 30 to 40 range and the decision between a bridge or a box culvert is not evident.

3. The Structure Survey Report indicates a multiple span structure in excess of 100 feet over a body of water.

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

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

If the preliminary bridge plans are required more than one year in advance of the final plan due date on the Structure Survey Report due to the unique needs of the project, the project manager should discuss this situation with the Consultant Design & Hydraulics Supervisor prior to submitting the Structure Survey Report. A note discussing the agreed upon schedule should then be attached to all copies of the Structure Survey Report so all parties are aware of the schedule. The Geotechnical Section is responsible for scheduling the borings.

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

The following figures are a sample of a Stream Crossing Structure Survey report; Figure 6.2-1, Figure 6.2-2, Figure 6.2-3, and Figure 6.2-4.

6.2.2 Preliminary Layout

6.2.2.1 General

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

For box culverts a preliminary drawing is usually not prepared. Information required to be submitted as a part of the survey report for a box culvert is usually sufficient to serve as a preliminary layout.

The drawings for preliminary layouts are on sheets having an overall width of 11 inches and an overall length of 17 inches.

ENGLISH - STREAM CROSSING STRUCTURE SURVEY REPORT Wisconsin Department of Transportation
DT1698 1/2006 (Replaces EB53)

☐ **Box Culvert** ☐ **Culvert Extension** ☐ Right ☐ Left ☒ **Stream Crossing** ☐ **Other**

Final Plan Due Date 3/1/1999	Preliminary Plan Due Date (N/A for Culverts) 10/1/1998	☒ Town of Winchester ☐ Village of ☐ City of	
New Structure Number B-70-139	Highway STH 110	County Winnebago	Design Project ID 6200-06-00
Aesthetics Level (For Levels 2, 3 & 4, Explain on Page 4) ☐ 1 ☒ 2 ☐ 3 ☐ 4			Construction Project ID 6200-06-71
Station 303 + 00	Section 35/36	Town 20N	Range 15E
Indicate Purpose ☒ Waterway ☐ Other (Describe)		Identify Stream (If Applicable) Arrowhead River	

Region Contact Person/Area Code with Telephone Number	Traffic Forecast Data			
D. Pauli 414-926-5672	Design Year	Average Daily Traffic (ADT)	Roadway Design Speed	Functional Class
Consultant Contact Person/Area Code with Telephone Number	2015	9150	100 mph	

Instructions for Structure Survey

In addition to this report, the following information shall be submitted.

1. **Small County Map** on which the location of proposed structure is shown in red and highway relocation, if any, in green.
2. **Plan and Profile Sheet** on proposed reference line of highway showing the following: (a) Ground line; (b) Finished grade line; (c) Profile grade line elevations at least every 100 feet for 1,000 feet each side of the structure; (d) Vertical curve control points; (e) Horizontal curve control points; (f) Curve data, including full SE and runoff distance.
3. **Contour Map** of the site drawn to a scale of not less than 1" = 20 feet with one-foot contours and showing the following (a) Existing highway and structure; (b) Proposed highway alignment and R/W; (c) Station numbers; (d) North arrow; (e) Buildings; (f) Underground facilities; (g) Other features which influence the design; (h) Recommended channel change; (i) Direction of stream flow; (j) Stations at end of existing structure; (k) Proposed structure and extent of riprap for consultant designed structures.
4. **Typical Roadway Cross Section** of proposed approaches showing the following: (a) Dimensions; (b) Slopes; (c) Type and width of surfacing or pavement; (d) Sidewalk, curb and gutter; (e) Subgrade and pavement thickness; (f) Clear zone width.
5. **Stream Cross Section** at upstream and downstream face of existing bridge and at one bridge length upstream and downstream. Surface water elevations at 1500 feet upstream and downstream of existing bridge.
6. **Original Photographs** of: (a) Existing structure; (b) Upstream and downstream structures; (c) Buildings within 100 feet of the proposed structure; (d) Unobstructed panoramic view looking upstream and downstream from proposed structure. *Air photo mosaics if available.*
7. **Proposed Location Map** showing structure location and number, one per structure when there are multiple structures on the project.
8. Attach a copy of the regulatory flood plain map (FEMA map) depicting the site.
9. For consultant designed structures - **Hydraulic Report** which may contain the following: (a) USGS quadrangle sheet showing proposed location, highway alignment and reach of river; (b) All available flood history, high water marks with date of occurrence, nature of flooding, damages and scour information; (c) Factors affecting water stages; (d) Navigation Clearance, for guidance in making report, see Chapter 8 of Bridge Design Manual; (e) Discussion of alternatives considered, factors influencing selection.
10. Attach a copy of DNR initial concurrence letter.

Figure 6.2-1

Structure Survey Report, Page 1 of 4

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

Table 6.3-1
Abbreviations

6.3.1.8 Nomenclature and Definitions

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

6.3.2 Plan Sheets

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

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

1. General Plan

2. Subsurface Exploration

3. Abutments

4. Piers

5. Superstructure and Superstructure Details

6. Railing and Parapet Details

Show all views looking up station.

6.3.2.1 General Plan (Sheet 1)

See the BOS web page, CADD Resource Files, for the latest sheet boarders to be used. Sheet borders are given for new bridges, rehabilitation projects and concrete box culverts. A superstructure replacement utilizing the existing substructure 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 with a required driving resistance of 180 tons * per pile as determined by the Modified Gates Dynamic Equation. Estimated 50' 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.

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

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

8. Estimated Quantities

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

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

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

9. General Notes

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

10. List of Drawings

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

11. Bench Marks

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

12. Title Block

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

13. Professional Seal

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

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

6.3.2.1.1 Plan notes for New Bridge Construction

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

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

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

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

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

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

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

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

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

10. The finished graded section shall be the upper limits of excavation for structures.

11. The upper limits of excavation for structures for the abutments shall be the bottom of slope protection.

12. 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.

13. 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.

14. 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.

15. 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

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

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

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

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

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

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

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

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

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

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

6.3.2.2 Subsurface Exploration

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

1. Plan View

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

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

2. Elevation

   Show a centerline profile of existing ground elevation.

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

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

6.3.2.3 Abutments

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

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

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

1. Plan View

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

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

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

2. Elevation

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

    b. Give vertical dimension of wing.

3. Wing Elevation

4. Body Section

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

5. Wing Sections

6. Bar Steel Listing and Detail

Use the following views where necessary:

7. Pile Plan & Splice Detail

8. View Showing Limits of Excavation and Backfill

9. Special Details for Utilities

10. Drainage Details

6.3.2.4 Piers

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

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

1. Plan View

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

2. Elevation

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

3. Footing Plan

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

4. Bar Steel Listing and Details

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

6. Cross Section thru Column and Pier Cap

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

6.3.2.5 Superstructure

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

6.3.2.5.1 All Structures

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

2. For girder bridges:
   Show the total dead load deflections, including composite dead load (without future wearing surface) acting on the composite section, at tenth points of each span. Distribute the composite dead load evenly to all girders and provide one deflection value for a typical interior girder. Chapter 17 – Superstructure-General illustrates three load cases for exterior girder design with raised sidewalks, cases that provide a conservative envelope to ensure adequate girder capacity. However, the above composite dead load distribution should be used for deflection purposes. For

prestressed concrete girders, the dead load deflection reported does not include the weight of the girder. See Chapter 24 – Steel Girder Structures for camber and blocking, top of steel elevation and deflection reporting criteria.

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

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

Deflection and camber values are to be reported to the nearest 0.1 inch.

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

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

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

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

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

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

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

6.3.2.5.2 Steel Structures

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

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

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

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

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

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

6.3.2.5.3 Railing and Parapet Details

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

6.3.3 Miscellaneous Information

6.3.3.1 Bill of Bars

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

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

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

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

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

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

6.3.3.2 Box Culverts

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

1. Plan View
2. Longitudinal section
3. Section thru box
4. Wing elevations
5. Section thru wings
6. Section thru cutoff wall
7. Vertical construction joint
8. Bar steel clearance details
9. Header details
10. North point, Bench mark, Quantities
11. Bill of bars, Bar details
12. General notes, List of drawings, Rip rap layout
13. Inlet nose detail on multiple cell boxes
14. Corner details

Bid items are excavation, concrete masonry, bar steel and rip rap. Non bid items are membrane waterproofing, filler and expansion bolts. In lieu of showing a contour map, show profile grade lines as described for Subsurface Exploration sheet.

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

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

6.3.3.3 Miscellaneous Structures

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

6.3.3.4 Standard Drawings

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

6.3.3.5 Insert Sheets

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

6.3.3.6 Change Orders and Maintenance Work

These plans are drawn on full size sheets.

6.3.3.7 Bench Marks

Bench mark caps are shown on all bridges and larger culverts. Locate the caps on a horizontal surface flush with the concrete. Show the location in close proximity to the Name Plate.

6.3.4 Checking Plans

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

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

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

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

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

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

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

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

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

* These items are added to the packet during construction.

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

1. All "void" material
2. All copies except one of preliminary drawings

3. Extra copies of plan and profile sheets

4. Preliminary computer design runs

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

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

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

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

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

4. Pile Driving Reports - HSI - Structures Development Section scans reports into HSI.

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. 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 the total quantity to the nearest cubic yard. Show unit quantities to the nearest 0.1 cubic yard adjusted so the total of the unit quantities equals 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 Structural Steel Carbon or Structural Steel HS

See 24.2.4.

6.4.7 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.

6.4.8 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.

6.4.9 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.

6.4.10 Preboring CIP Concrete Piling or Steel Piling

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

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

Record the type, quantity is a Lump Sum.

6.4.12 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.

6.4.13 Riprap Medium, Riprap Heavy or Grouted Riprap, Riprap Light

Record this quantity to the nearest 5 cubic yards.

6.4.14 Pile Points

When recommended in soils report. Bid as each.

6.4.15 Floordrains Type GC or Floordrains Type H

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

6.4.16 Cofferdams (Structure)

Lump Sum

6.4.17 Rubberized Membrane Waterproofing

Record the quantity to the nearest square yard.

6.4.18 Expansion Device (Structure)

Record this quantity in lump sum.

6.4.19 Electrical Work

Refer to Standard Construction Specifications for bid items.

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

Record this quantity in feet.

6.4.21 Preparation Decks Type 1 or Preparation Decks Type 2

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

6.4.22 Cleaning Decks

Record this quantity to the nearest square yard.

6.4.23 Joint Repair

Record this quantity to the nearest square yard.

6.4.24 Concrete Surface Repair

Record this quantity to the nearest square foot.

6.4.25 Full-Depth Deck Repair

Record this quantity to the nearest square yard.

6.4.26 Concrete Masonry Overlay Decks

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

6.4.27 Removing Old Structure STA. XX + XX.XX

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

6.4.28 Anchor Assemblies for Steel Plate Beam Guard

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

6.4.29 Steel Diaphragms (Structure)

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

6.4.30 Welded Stud Shear Connectors X -Inch

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

6.4.31 Concrete Masonry Seal

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

6.4.32 Geotextile Fabric Type

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

6.4.33 Masonry Anchors Type L No. Bars

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

6.4.34 Piling Steel Sheet Permanent Delivered or Piling Steel Sheet Permanent Driven

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

6.4.35 Piling Steel Sheet Temporary

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

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

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

6.4.36 Temporary Shoring

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

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

6.4.37 Concrete Masonry Deck Patching

(Deck preparation areas) x 2" deck thickness.

6.4.38 Sawing Pavement Deck Preparation Areas

Use 10 lineal feet per S.Y. of Preparation Decks.

6.4.39 Removing Bearings

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

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

The need for structures is determined during the Preliminary Site Survey and recorded in the Concept Definition or Work Study Report. On Federal (FHWA) or State Aid Projects completed Structure Survey Reports and plans are submitted to the Structures Design Section with a copy forwarded to the Regional Office for approval prior to construction. Structure and project numbers are assigned by the Regional Offices. In preparation of the structural plans, the appropriate specifications and details recommended by the Structures Design Section are to be used. If the consultant elects to modify or use details other than recommended, approval is required prior to their incorporation into the final plans.

On all Federal or State Aid Projects involving Maintenance work, the Concept Definition or Work Study Report, the preliminary and final bridge reconstruction plans shall be submitted to the Structures Design Section for review.

Consultants desiring eligibility to perform engineering and related services on WisDOT administered structure projects must have on file with the Bureau of Structures, an electronic copy of their current Quality Assurance/Quality Control (QA/QC) plan and procedures. The QA/QC plan and procedures shall include as a minimum:

- Procedures to detect and correct bridge design errors before the design plans are made final.
- A means for verifying that the appropriate design calculations have been performed, that the calculations are accurate, and that the capacity of the load-carrying members is adequate with regard to the expected service loads of the structure.
- A means for verifying the completeness, constructability and accuracy of the structure plans.
- Verification that independent checks, reviews and ratings were performed.

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

6.5.1 Approvals, Distribution, and Work Flow

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

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

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

Table 6.5-1
Approvals, Distribution and Work Flow

6.5.2 Preliminary Plan Requirements

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

1. Structure Survey Report.

2. Preliminary Drawings.

3. Log Borings shown on the Subsurface Exploration Drawing which must be submitted now and can be included with the Final Plans.

4. Evaluation Report of Borings with Values for End Bearing and/or Skin Friction.

5. Contour Map.

6. Typical Section for Roadway Approaches.

7. Plan and Profile of Approach Roadways.

8. Hydraulic Report (see Chapter 8 - Hydraulics) is required for Stream Crossing Structures.

9. County Map showing Location of New and/or Existing Structures.

10. Any other information or Drawings which may influence Location, Layout or Design of Structure.

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

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

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

6.5.3 Final Plan Requirements

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

1. Final Drawings.

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

2. Design and Quantity Computations

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

4. QA/QC Verification Sheet

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

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

6.5.4 Design Aids & Specifications

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

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

Bridge Manual
Highway Structures Information System (HSI)
Insert sheets
Standard details
Posted bridge map
Standard bridge CADD files

Structure survey reports and check lists
Structure costs
Structure Special Provisions

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

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

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

http://bridges.transportation.org

http://www.arema.org

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9.1 General

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

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

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

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

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

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 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

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11.1 General

11.1.1 Overall Design Process

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

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

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

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

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

11.1.2 Foundation Type Selection

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

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

$$R_p = 9 S_u A_p$$

Where:

R_p = Point resistance capacity (tons)

S_u = Undrained shear strength of the cohesive soil near the pile base (tsf)

A_p = Pile end area (feet^2)

This equation represents the maximum value of point resistance for cohesive soil. This value is often assumed to be zero because substantial movement of the pile tip (1/10 of the pile diameter) is needed to mobilize point resistance capacity. This amount of tip movement seldom occurs after installation.

A point resistance (or end bearing) pile surrounded by soil is not a structural member like a column. Both experience and theory demonstrate that there is no danger of a point resistance pile buckling due to inadequate lateral support if it is surrounded by even the very softest soil. Therefore, pile stresses can exceed column stresses. Exposed pile bent piles may act as structural columns.

11.3.1.15.3 Group Capacity

The nominal resistance capacity of pile groups may be less than the sum of the individual nominal resistances of each pile in the group for friction piles founded in cohesive soil. For pile groups founded in cohesive soil, the pile group must be analyzed as an equivalent pier for block failure in accordance with **LRFD [10.7.3.9]**. WisDOT no longer accepts the Converse-Labarre method of analysis to account for group action. If the pile group is tipped in a firm stratum overlying a weak layer, the weak layer should be checked for possible punching failure in accordance with **LRFD [10.6.3.1.2a]**. Experience in Wisconsin indicates that in most thixotropic clays where piles are driven to a hammer bearing as determined by dynamic formulas, pile group action is not the controlling factor to determine pile resistance capacity. For pile groups in sand, the sum of the nominal resistance of the individual piles always controls the group resistance.

11.3.1.16 Lateral Load Resistance

Structures supported by single piles or pile groups are frequently subjected to lateral forces from lateral earth pressure, live load forces, wave action, ice loads and wind forces. Piles subjected to lateral forces must be designed to meet combined stress and deflection criteria to prevent impairment or premature failure of the foundation or superstructure. To solve the soil-structure interaction problems, the designer must consider the following:

- Pile group configuration.

- Pile stiffness.

- Degree of fixity at the pile connection with the pile footing.
- Maximum bending moment induced on the pile from the superstructure load and moment distribution along the pile length.
- Probable points of fixity near the pile tip.
- Soil response (P-y method) for both the strength and service limit states.
- Pile deflection permitted by the superstructure at the service limit state.

If a more detailed lateral load investigation is desired, a P-y analysis is typically performed using commercially available software such as COM624P, FB Multi-Pier or L-Pile. A resistance factor is not applied to the soil response when performing a P-y analysis using factored loads since the soil response represents a nominal (ultimate) condition. For a more detailed analysis of lateral loads and displacements, refer to the listed FHWA design references or a geotechnical engineering book.

WisDOT policy item:

A detailed analysis is required for the lateral resistance of piles used in A3 and A4 abutments.

11.3.1.17 Other Design Considerations

Several other topics should be considered during design, as presented below.

11.3.1.17.1 Downdrag Load

Negative shaft resistance (downdrag) results in the soil adhesion forces pulling down the pile instead of the soil adhesion forces resisting the applied load. This can occur when settlement of the soil through which the piling is driven takes place. It has been found that only a small amount of settlement is necessary to mobilize these additional pile (drag) loads. This settlement occurs due to consolidation of softer soil strata caused by such items as increased embankment loads (due to earth fill) or a lowering of the existing ground water elevation. The nominal pile resistance available to support structure loads plus downdrag shall be estimated by considering only the positive skin and tip resistance below the lowest layer acting to produce negative skin resistance. When this condition is present, the designer may provide time to allow consolidation to occur before driving piling or **LRFD [10.7.3.8.6]** may be used to estimate the available pile resistance to withstand the downdrag plus structure loads. Other alternatives are to preauger the piling, drive the pile to bearing within a permanent pipe sleeve that is placed from the base of the substructure unit to the bottom of the soft soil layer(s), coat the pile with bitumen above the compressible soil strata or use proprietary materials to encase the piles (within fill constructed after the piling is installed). The Department has experienced problems with bitumen coatings.

The factored axial compression resistance values given for H-piles in Table 11.3-5 are conservative and based on Departmental experience to avoid overstressing during driving. For H-piles in end bearing, loading from downdrag is allowed in addition to the normal pile loading since this is a post-driving load. Use the values given in Table 11.3-5 and design piling as usual. Additionally, up to 45, 60, and 105 tons downdrag for HP 10x42, HP 12x53, and HP 14x73 piles respectively is allowed.

11.3.1.17.2 Lateral Squeeze

Lateral squeeze as described in **LRFD [10.7.2.6]** occurs when pile supported abutments are constructed on embankments and/or MSE walls over soft soils. Typically, the piles are installed prior to completion of the embankment and/or MSE wall, and therefore are potentially subject to subsurface soil instability. If the embankment and/or MSE wall has a marginal factor of safety with regards to slope stability, then lateral squeeze has the potential to laterally deflect the piles and tilt the abutment. Typically, if the shear strength of the subsurface soil is less than the height of the embankment times the unit weight of the embankment divided by three, then damage from lateral squeeze could be expected.

If this is a potential problem, the following are the recommended solutions from the *FHWA Design and Construction of Driven Piles Manual*:

1. Delay installation of abutment piling until after settlement has stabilized (best solution).

2. Provide expansion shoes large enough to accommodate the movement.

3. Use steel H-piles strong enough and rigid enough to provide both adequate strength and deflection control.

4. Use lightweight fill to reduce driving forces.

11.3.1.17.3 Uplift Resistance

Uplift forces may also be present, both permanently and intermittently, on a pile system. Such forces may occur from hydrostatic uplift or cofferdam seals, ice uplift resulting from ice grip on piles and rising water, wind uplift due to pressures against high structures or frost uplift. In the absence of pulling test data, the resistance factors from Table 11.3-1 should be used to determine static uplift resistance. Generally, the type of pile with the largest perimeter is the most efficient in resisting uplift forces.

11.3.1.17.4 Pile Setup and Relaxation

The nominal resistance of a deep foundation may change over time, particularly for driven piles. The nominal resistance may increase (setup) during dissipation of excess pore pressure, which developed during pile driving, as soil particles reconsolidate after the soil has been remolded during driving. The shaft resistance may decrease (relaxation) during dissipation of negative pore pressure, which was induced by physical displacement of soil during driving. If the potential for soil relaxation is significant, a non-displacement pile is preferred over a displacement type pile. Relaxation may also occur as a result of a deterioration of the bearing stratum following driving-induced fracturing, especially for point-bearing piles founded on non-durable bedrock. Relaxation is generally associated with densely compacted granular material.

Pile setup has been found to occur in some fine-grained soil in Wisconsin. Pile setup should not be included in pile design unless pre-construction load tests are conducted to determine site-specific setup parameters. The benefits of obtaining site-specific setup parameters could include shortening friction piles and reducing the overall foundation cost. Pile driving resistance would need to be determined at the end of driving and again later after pore pressure dissipation. Restrike tests involve additional taps on a pile after the pile has been driven and a waiting period (generally 24 to 72 hours) has elapsed. The dynamic monitoring analysis are used to predict resistance capacity and distribution over the pile length.

CAPWAP(CAse Pile Wave Analysis Program) is a signal matching software. CAPWAP uses dynamic pile force and velocity data to discern static and dynamic soil resistance, and then estimate static shaft and point resistance for driven pile. Pile top force and velocity are calculated based on strain and acceleration measurements during pile driving, with a pile driving analyzer (PDA). CAPWAP is based on the wave equation model which characterizes the pile as a series of elastic beam elements, and the surrounding soil as plastic elements with damping (dynamic resistance) and stiffness (static resistance) properties.

Typically, a test boring is drilled and a static load test is performed at test piles where pile setup properties are to be determined. Typical special provisions have been developed for use on projects incorporating aspects of pile setup. Pile setup is discussed in greater detail in FHWA Publication NHI-05-042, *Design and Construction of Driven Pile Foundations*.

Restrike tests with an impact hammer can be used to identify change in pile resistance due to pile setup or relaxation. Restrike is typically performed by measuring pile penetration during the first 10 blows by a warm hammer. Due to setup, it is possible that the hammer used for initial driving may not be adequate to induce pile penetration and a larger hammer may be required to impart sufficient energy for restrike tests. Only warm hammers should be used for restrikes by first applying at least 20 blows to another pile.

Restrike tests with an impact hammer must be used to substantiate the resistance capacity and integrity of pile that is initially driven with a vibratory hammer. Vibratory hammers may be used with approval of the engineer. Other than restrikes with an impact hammer, no formula exists to reliably predict the resistance capacity of a friction pile that is driven with a vibratory hammer.

11.3.1.17.5 Drivability Analysis

In order for a driven pile to develop its design geotechnical resistance, it must be driven into the ground without damage. Stresses developed during driving often exceed those developed under even the most extreme loading conditions. The critical driving stress may be either compression, as in the case of a steel H-pile, or tension, as in the case of a concrete pile.

Drivability is treated as a strength limit state. The geotechnical engineer will perform the evaluation of this limit state during design based on a preliminary dynamic analysis using wave equation techniques. These techniques are used to document that the assumed pile driving hammers are capable of mobilizing the required nominal (ultimate) resistance of the pile at driving stress levels less than the factored driving resistance of the pile. Drivability can often be the controlling strength limit state check for a pile foundation. This is especially true for high capacity piles driven to refusal on rock.

Drivability analysis is required by **LRFD [10.7.8]**. A drivability evaluation is needed because the highest pile stresses are usually developed during driving to facilitate penetration of the pile to the required resistance. However, the high strain rate and temporary nature of the loading during pile driving allow a substantially higher stress level to be used during installation than for service. The drivability of candidate pile-hammer-system combinations can be evaluated using wave equation analyses.

As stated in the 2004 FHWA Design and Construction of Driven Pile Foundations Manual:

> "The wave equation does not determine the capacity of the pile based on soil boring data. The wave equation calculates a penetration resistance for an assumed ultimate capacity, or conversely it assigns estimated ultimate capacity to a pile based upon a field observed penetration resistance."
>
> "The accuracy of the wave equation analysis will be poor when either soil model or soil parameters inaccurately reflect the actual soil behavior, and when the driving system parameters do not represent the state of maintenance of hammer or cushions."

The following presents potential sources of wave equation errors.

- Hammer Data Input, Diesel Hammers
- Cushion Input
- Soil Parameter Selection

LRFD [C10.7.8] states that the local pile driving results from previous drivability analyses and historical pile driving experience can be used to refine current drivability analyses. WisDOT recommends using previous pile driving records and experience when performing and evaluating drivability analyses. These correlations with past pile driving experience allow modifications of the input values used in the drivability analysis, so that results agree with past construction findings.

Driving stress criteria are specified in the individual LRFD material design sections and include limitations of unfactored driving stresses in piles based on the following:

- Yield strength in steel piles, as specified in **LRFD [6.4.1]**
- Ultimate compressive strength of the gross concrete section, accounting for the effective prestress after losses for prestressed concrete piles loaded in tension or compression, as specified in **LRFD [5.7.4.4]**

Though there are a number of ways to assess the drivability of a pile, the steps necessary to perform a drivability analysis are typically as follows:

1. Estimate the total resistance of all soil layers. This may include layers that are not counted on to support the completed pile due to scour or potential downdrag, but will have to be driven through. WisDOT recommends using the values for quake and damping provided in the FHWA Design and Construction of Driven Pile Foundations Manual.

   In addition, the soil resistance parameters should be reduced by an appropriate value to account for the loss of soil strength during driving. The following table provides some guidelines based on Table 9-19 of the FHWA Design and Construction of Driven Pile Foundations Manual:

Soil Type	Recommended Soil Set Up Factor[1]	Percentage Loss of Soil Strength during Driving
Clay	2.0	50 percent
Silt – Clay	1.5[2]	33 percent
Silt	1.5	33 percent
Sand – Clay	1.5	33 percent
Sand – Silt	1.2	17 percent
Fine Sand	1.2	17 percent
Sand	1.0	0 percent
Sand - Gravel	1.0	0 percent
Notes: 1. Confirmation with local experience recommended 2. The value of 1.5 is higher than the FHWA Table 9-19 value of 1.0 based upon WisDOT experience.		

Table 11.3-4
Soil Resistance Factors

Incorporation of loss of soil strength and soil set-up should only be accounted for in the pile drivability analyses. Typically, WisDOT does not include set-up in static pile design analyses.

2. Select a readily available hammer. The following hammers have been used by Wisconsin Bridge Contractors: Delmag D-12-42, Delmag D-12-32, Delmag D-12, Delmag D-15, Delmag D-16-32, Delmag D-19, Delmag D-19-32, Delmag D-19-42, Delmag D-25, Delmag D-30-32, Delmag D-30, Delmag D-36, MKT-7, Kobe K-13, Gravity Hammer 5K.

3. Model the driving system, soil and pile using a wave equation program. The driving system generally includes the pile-driving hammer, and elements that are placed between the hammer and the top of pile, which include the helmet, hammer cushion, and pile cushion (concrete piles only). Pile splices are also modeled. Compute the driving stress using the drivability option for the wave equation, which shows the pile compressive stress an blow counts versus depth for the given soil profile.

4. Determine the permissible driving stress in the pile. During the design stage, it is often desirable to select a lower driving stress than the maximum permitted. This will allow the contractors greater flexibility in hammer selection. WisDOT generally limits driving stress to 90 percent of the steel yield strength

5. Evaluate the results of the drivability analysis to determine a reasonable blow count (that is, ranges from 25 blows per foot to 120 blows per foot) associated with the permissible driving stress.

The goal of the drivability study is to evaluate the potential for excessive driving stresses and to determine that the pile/soil system during driving will result in reasonable blow counts. The drivability study is not intended to evaluate the ultimate pile capacity or establish plan lengths. If the wave equation is used to set driving criteria, then contact the Bureau of Technical Services, Foundation and Pavement Unit to discuss the proper procedures.

11.3.1.17.6 Scour

During design, estimated pile lengths are increased to compensate for scour loss. The scour depth is estimated and used to compute the estimated shaft resistance that is lost over the scour depth (exposed pile length). The required pile length is then increased to compensate for the resistance capacity that is lost due to scour. The pile length is increased based on the following equation:

$$R_n = R_{n\text{-stat}} + R_{n\text{-scour}}$$

Where:

R_n = Nominal shaft resistance capacity, adjusted for scour effect (tons)

$R_{n\text{-stat}}$ = Nominal shaft resistance based on static analysis, without scour consideration (tons)

$R_{n\text{-scour}}$ = Nominal shaft resistance lost (negative value) over the exposed pile length due to scour (tons)

WisDOT policy item:

If there is potential for scour at a site, it is WisDOT policy to account for the loss of pile resistance from the material within the scour depth. The designer must not include any resistance provided by this material when determining the nominal pile resistance. Since the material within the scour depth may be present during pile driving operations, the additional resistance provided by this material shall be included when determining the required driving resistance. The designer should also consider minimum pile tip elevation requirements.

11.3.1.17.7 Typical Pile Resistance Values

Table 11.3-5 shows the typical pile resistance values for several pile types utilized by the Department. The table shows the Nominal Axial Compression Resistance (Pn), which is a function of the pile materials, the Factored Axial Compression Resistance (Pr), which is a function of the construction procedures, and the Required Driving Resistance, which is a function of the method used to measure pile capacity during installation. The bridge designer uses the Factored Axial Compression Resistance to determine the number and spacing of the piles. The Required Driving Resistance is placed on the plans. See 6.3.2.1-7 for details regarding plan notes.

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

Table 11.3-5
Typical Pile Resistance Values

Notes

1. NA – not applicable
2. For CIP Piles: Pn = 0.8 (0.85 * f’c * Ag + fy * As) **LRFD [5.5.4.2.1]**. Neglecting the steel shell, equation reduces to 0.68 * f’c * Ag.

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

3. For H-Piles: Pn = 0.66^{λ} * Fy * As (λ = 0 for piles embedded below the substructure)

   fy = yield strength of steel = 50,000 psi

4. Pr = ϕ * Pn

   ϕ = 0.75 (**LRFD [5.5.4.2.1]** for axial compression concrete)

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

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

   - Rn_{dyn} = Pr / φ_{dyn}

     φ_{dyn} = 0.5 (**LRFD [Table 10.5.5.2.3-1]** for construction driving criteria using modified Gates dynamic formula)

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

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

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

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

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

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

11.3.1.18 Construction Considerations

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

11.3.1.18.1 Pile Hammers

Pile driving hammers are generally powered by compressed air, steam pressure or diesel units. The diesel hammer, a self-contained unit, is the most popular due to its compactness and adoption in most construction codes. Also, the need for auxiliary power is eliminated and the operation cost is nominal. Vibratory and sonic type hammers are employed in special cases where speed of installation is important and/or noise from impact is prohibited. The vibrating hammers convert instantly from a pile driver to a pile extractor by merely tensioning the lift line.

Pile hammers are raised and allowed to fall either by gravity or with the assistance of power. If the fall is due to gravity alone, the hammer is referred to as single-acting. The single-acting hammer is suitable for all types of soil but is most effective in penetrating heavy clays. The major disadvantage is the slow rate of driving due to the relatively slow rate of blows from 50 to 70 per minute. Wisconsin construction specifications call for a minimum hammer weight depending on the required final bearing value of the pile being driven. In order to avoid damage to the pile, the fall of the gravity hammer is limited to 10 feet.

If power is added to the downward falling hammer, the hammer is referred to as double-acting. This type of hammer works best in sandy soil but also performs well in clay. Double-acting hammers deliver 100 to 250 blows per minute, which increases the rate of driving considerably over the single-acting hammers. Wisconsin construction specifications call for a rated minimum energy of 15 percent of the required bearing of the pile. A rapid succession of blows at a high velocity can be extremely inefficient, as the hammer bounces on heavy piles.

Differential-acting hammers overcome the deficiencies found with both single- and double-acting hammers by incorporating higher frequency of blows and more efficient transfer of energy. The steam cycle, which is different from that of any other hammer, makes the lifting area under the piston independent of the downward thrusting area above the piston. Sufficient force can be applied for lifting and accelerating these parts without affecting the dead weight needed to resist the reaction of the downward acceleration force. The maximum delivered energy per blow is the total weight of the hammer plus the weight of the downward steam force times the length of the stroke.

The contractor’s selection of the pile hammer is generally dependent on the following:

- The hammer weight and rated energy are selected on the basis of supplying the maximum driving force without damaging the piles.
- The hammer types dictated by the construction specification for the given pile type.
- The hammer types available to the contractor.
- Special situations, such as sites adjacent to existing buildings, that require consideration of vibrations generated from the driving impact or noise levels. In these instances, reducing the hammer size or choosing a double-acting hammer may be preferred over a single-acting hammer. Impact hammers typically cause less ground vibration than vibratory hammers.

- The subsurface conditions at the site.
- The required final resistance capacity of the pile.

WisDOT specifications require the heads of all piling to be protected by caps during driving. The pile cap serves to protect the pile, as well as modulate the blows from the hammer which helps eliminate large inefficient hammer forces. When penetration-per-blow is used as the driving criteria, constant cap-block material characteristics are required. The cap-block characteristics are also assumed to be constant for all empirical formula computations to determine the rate of penetration equivalent to a particular dynamic resistance.

11.3.1.18.2 Driving Formulas

Formulas used to estimate the bearing capacity of piles are of four general types – empirical, static, dynamic and wave equation.

Empirical formulas are based upon tests under limited conditions and are not suggested for general use.

Static formulas are based on soil stresses and try to equate shaft resistance and point resistance to the load-bearing capacity of the piles.

Dynamic pile driving formulas assume that the kinetic energy imparted by the pile hammer is equal to the nominal pile resistance plus the energy lost during driving, starting with the following relationship:

$$\text{Energy input} = \text{Energy used} + \text{Energy lost}$$

The energy used equals the driving resistance multiplied by the pile movement. Thus, by knowing the energy input and estimating energy losses, driving resistance can be calculated from observed pile movement. Numerous dynamic formulas have been proposed. They range from the simpler Engineering News Record (ENR) Formula to the more complex Hiley Formula. A modified Engineering News Formula was previously used by WisDOT to determine pile resistance capacity during installation. All new designs shall use the modified Gates or WAVE equation for determining the required driving resistance.

The following modified FHWA-Gates Formula is used by WisDOT:

$$R_R = \varphi_{dyn} R_{ndr} = \varphi_{dyn}\left(0.875\left(E_d\right)^{0.5}\log_{10}\left(10/s\right) - 50\right)$$

Where:

R_R = Factored pile resistance (tons)

φ_{dyn} = Resistance factor = 0.5 **LRFD [Table 10.5.5.2.3-1]**

R_{ndr} = Nominal pile resistance measured during pile driving (tons)

E_d = Energy delivered by the hammer per blow (lb-foot)

s = Average penetration in inches per blow for the final 10 blows (inches/blow)

Because of the difficulty of evaluating the many energy losses involved with pile driving, these dynamic formulas can only approximate pile driving resistance. These approximate results can be used as a safe means of determining pile length and bearing requirements. Despite the obvious limitations, the dynamic pile formulas take into account the best information available and have considerable utility to the engineer in securing reasonably safe and uniform results over the entire project.

The wave equation can be used to set driving criteria to achieve a specified pile bearing capacity (contact the Bureau of Technical Services, Geotechnical Unit prior to using the wave equation to set the driving criteria). The wave equation is based upon the theory of longitudinal wave transmission. This theory, proposed by Saint Venant a century ago, did not receive widespread use until the advent of computers due to its complexity. The wave equation can predict impact stresses in a pile during driving and estimate static soil resistance at the time of driving by solving a series of simultaneous equations. An advantage of this method is that it can accommodate any pile shape, as well as any distribution of pile shaft resistance and point resistance. The effect of the hammer and cushion block can be included in the computations.

Dynamic monitoring is performed by a Pile Driving Analyzer (PDA). WisDOT uses the PDA to evaluate the driving criteria, which is set by a wave equation analysis, and in an advisory capacity for evaluating if sufficient pile penetration is achieved, if pile damage has occurred or if the driving system is performing satisfactorily.

The PDA provides a method of dynamic pile testing both for pile design and construction control. Testing is accomplished during pile installation by attaching reusable strain transducers and accelerometers directly on the pile. Piles can be tested while being driven or during restrike. The instrumentation mounted on the pile allows the measurement of force and acceleration signals for each hammer blow. This data is transmitted to a small field computer for processing and recording on a magnetic data tape for a permanent record. Data is also fed into an oscilloscope to observe the dynamic response of the pile to driving. Calculations made by the computer based upon one-dimensional wave mechanics provide an immediate printout of maximum stresses in the pile, energy transmitted to the pile and a prediction of the nominal axial resistance of the pile for each hammer impact. Monitoring of the force and velocity wave traces with the oscilloscope during driving also enables detection of any structural pile damage that may have occurred. Printouts of selected force and velocity wave traces are also made to provide additional testing documentation. The PDA can be used on all types of driven piles with any impact type of pile-driving hammer but is most often used for displacement piles.

11.3.1.18.3 Field Testing

Test piles are employed at a project site for two purposes:

- For test driving, to determine the length of pile required prior to placing purchasing orders.
- For load testing, to verify actual pile capacity versus design capacity for nominal axial resistance.

11.3.1.18.3.1 Installation of Test Piles

Test piles are not required for spliceable types of piles. Previous experience indicates that contractors typically order total plan quantities for cast-in-place or steel H-piling in 60-foot lengths. The contractor uses one of the driven structure piles as a test pile at each designated location.

Test piling should be driven near the location of a soil boring where the soil characteristics are known and representative of the most unfavorable conditions at the site. The test pile must be exactly the same type and dimension as the piles to be used in the construction and installed by the same equipment and manner of driving. A penetration record is kept for every 1 foot of penetration for the entire length of pile. This record may be used as a guide for future pile driving on the project. Any subsequent pile encountering a smaller resistance is considered as having a smaller nominal resistance capacity than the test pile.

11.3.1.18.3.2 Static Load Tests

A pile load test is usually conducted to furnish information to the geotechnical engineer to develop design criteria or to obtain test data to substantiate nominal resistance capacity for piles. A load test is the only reliable method of determining the nominal bearing resistance of a single pile, but it is expensive and can be quite time consuming. The decision to embark on an advance test program is based upon the scope of the project and the complexities of the foundation conditions. Such test programs on projects with large numbers of displacement piling often result in substantial savings in foundation costs, which can more than offset the test program cost. WisDOT has only performed a limited number of pile load tests on similar type projects.

Pile load testing generally involves the application of a direct axial load to a single vertical pile. However, load testing can involve uplift or axial tension tests, lateral tests applied horizontally, group tests or a combination of these applied to battered piles. Most static test loads are applied with hydraulic jacks reacting against either a stable loaded platform or a test frame anchored to reaction piles.

The basic information to be developed from the pile load test is usually the deflection of the pile head under the test load. Movement of the head is caused by elastic deformation of the piles and the soil. Soil deformation may cause undue settlement and must be guarded against. The amount of deformation is the significant value to be obtained from load tests, rather than the total downward movement of the pile head. Load tests are typically performed by loading to a given deflection value.

It is impractical to test every pile on a project. Therefore, test results can be applied to other piles or pile groups providing that the following conditions exist:

- The other piles are of the same type, material and size as the test piles.
- Subsoil conditions are comparable to those at the test pile locations.
- Installation methods and equipment used are the same as, or comparable to, those used for the test piles.
- Piles are driven to the same penetration depth or resistance or both as the test piles to compensate for variations in the vertical position and density of the bearing strata.

11.3.2 Drilled Shafts

11.3.2.1 General

Drilled shafts are generally large diameter, cast-in-place, open ended, cased concrete piles which are designed to carry extremely heavy loads. Drilled shafts can be the most economical foundation alternative at sites where foundation loads are carried to bearing on dense strata or bedrock. They are also cost effective in water crossings with very shallow bedrock, where cofferdams are difficult or expensive to construct, and where high overturning moments must be resisted.

Drilled shafts are installed by removing soil and rock using drilling methods or other excavation techniques and constructing the foundation element in the excavated hole. The excavated hole may be supported using temporary or permanent casing, drilling slurry or other methods. The hole is then filled with a reinforcement cage and cast-in-place concrete. Drilled shafts are non-displacement elements since the soil volume required for the element is physically removed prior to installation. Thus the effective normal stress adjacent to the pile remains unchanged or is reduced (due to expansion of the soil into the hole before insertion/construction of the load bearing element), and the soil properties and pore water pressure adjacent to the foundation elements are not significantly impacted.

Because drilled shafts do not require a hammer for installation and do not displace the soil, they typically have much less impact on adjacent structures. Depending on the excavation technique used, they can penetrate significant obstructions. Because the method of construction often allows a decrease in the effective stress immediately adjacent to and beneath the tip of the foundation element, the resistance developed will often be less than an equivalently sized driven pile.

Drilled shafts are generally considered fixed to the substructure unit if the reinforcing steel from the shaft is fully developed within the substructure unit.

Drilled shafts vary in diameter from approximately 2.5 to 10 feet. Drilled shafts with diameters greater than 6 feet are generally referred to as piers. Shafts may be designed to transfer load to the bearing stratum through side friction, point-bearing or a combination of both. The drilled shaft may be cased or uncased, depending on the subsurface conditions and depth of bearing.

Drilled shafts have been used on only a small number of structures in Wisconsin. For unusual site conditions, the use of drilled shafts may be advantageous. Design methodologies for drilled shafts can be found in FHWA Publication IF-99-025, *Drilled Shafts: Construction Procedures and Design Methods*.

Strength limit states for drilled shafts are evaluated in the same way as for driven piles. Drivability is not required to be evaluated. The structural resistance of drilled shafts is evaluated in accordance with **LRFD [5.7 and 5.8]**. This includes evaluation of axial resistance, combined axial and flexure, shear and buckling. It is noted that the critical load case for combined axial and flexure may be a load case that results in the minimum axial load or tension.

11.3.2.2 Resistance Factors

Resistance factors for drilled shafts are presented in Table 11.3-6 and are selected based on the method used to determine the nominal (ultimate) resistance capacity of the drilled shaft. The design intent is to adjust the resistance factor based on the reliability of the method used to determine the nominal shaft resistance. As with driven piles, the selection of a geotechnical resistance factor should be based on the intended method of resistance verification in the field. Because of the cost and difficulty associated with testing drilled shafts, much more reliance is placed on static analysis methods.

Condition/Resistance Determination Method				Resistance Factor
Static Analysis – Used in Design Phase	Nominal Resistance of Single-Drilled Shaft in Axial Compression, φ_{stat}	Shaft Resistance in Clay	Alpha Method	0.45
		Point Resistance in Clay	Total Stress	0.40
		Shaft Resistance in Sand	Beta Method	0.55
		Point Resistance in Sand	O'Neill and Reese	0.50
		Shaft Resistance in IGMs	O'Neill and Reese	0.60
		Point Resistance in IGMs	O'Neill and Reese	0.55
		Shaft Resistance in Rock	Horvath and Kenney O'Neill and Reese	0.55
			Carter and Kulhawy	0.50
		Point Resistance in Rock	Canadian Geotech. Soc. Pressuremeter Method O'Neill and Reese	0.50
	Block Failure, φ_{bl}	Clay		0.55
	Uplift Resistance of Single-Drilled Shaft, φ_{up}	Clay	Alpha Method	0.35
		Sand	Beta Method	0.45
		Rock	Horvath and Kenney Carter and Kulhawy	0.40
	Group Uplift Resistance, φ_{ug}	Sand and Clay		0.45
	Horizontal Geotechnical Resistance of Single Shaft or Pile Group	All Soil Types and Rock		1.0

Table 11.3-6

Geotechnical Resistance Factors for Drilled Shafts **LRFD [Table 10.5.5.2.4-1]**

For drilled shafts, the base geotechnical resistance factors in Table 11.3-6 assume groups containing two to four shafts, which are slightly redundant. For groups containing at least four elements, the base geotechnical resistance factors in Table 11.3-6 should be increased by 20%. WisDOT generally uses 4 or more shafts per substructure unit.

WisDOT policy item:

WisDOT policy requires a multi-column bent to be designed as a redundant rigid frame. Hence when a bent contains at least 4 columns then the resistance factors in Table 11.3-6 should be increased by 20 percent.

Resistance factors for structural design of drilled shafts are obtained from **LRFD [5.5.4.2.1]**.

11.3.2.3 Bearing Resistance

Most drilled shafts provide geotechnical resistance in both end bearing and side friction. Because the rate at which side friction mobilizes is usually much higher than the rate at which end bearing mobilizes, past design practice has been to ignore either end bearing for shafts with significant sockets into the bearing stratum or to ignore skin friction for shafts that do not penetrate significantly into the bearing stratum. This makes evaluation of the geotechnical resistance slightly more complex, because in most cases it is not suitable to simply add the nominal (ultimate) end bearing resistance and the nominal side friction resistance in order to obtain the nominal axial geotechnical resistance.

When computing the nominal geotechnical resistance, consideration must be given to the anticipated construction technique and the level of construction control. If it is anticipated to be difficult to adequately clean out the bottom of the shafts due to the construction technique or subsurface conditions, the end bearing resistance may not be mobilized until very large deflections have occurred. Similarly, if construction techniques or subsurface conditions result in shaft walls that are very smooth or smeared with drill cuttings, side friction may be far less than anticipated.

Because these resistances mobilize at different rates, it may be more appropriate to add the ultimate end bearing to that portion of the side resistance remaining at the end of bearing failure. Or it may be more appropriate to add the ultimate side resistance to that portion of the end bearing mobilized at side resistance failure. Note that consideration of deflection, which is a service limit state, may control over the axial geotechnical resistance since displacements required to mobilize the ultimate end bearing can be excessive. More detailed discussion of design parameters is provided in Appendices C and D of FHWA Publication IF-99-025, *Drilled Shafts: Construction Procedures and Design Methods*.

11.3.2.3.1 Shaft Resistance

The shaft resistance is estimated by summing the friction developed in each stratum. When drilled shafts are socketed in rock, the shaft resistance that is developed in soil is generally ignored to satisfy strain compatibility. The following analysis methods are typically used to compute the static shaft resistance in soil and rock:

- Alpha method for cohesive soil, as specified in **LRFD [10.8.3.5.1]**
- Beta method (β-method) for cohesionless soil, as specified in **LRFD [10.8.3.5.2]**
- Horvath and Kenny method for rock, as specified in **LRFD [10.8.3.5.4]**

11.3.2.3.2 Point Resistance

The following analysis methods are typically used to compute the static shaft resistance in soil:

- Alpha method for cohesive soil, as specified in **LRFD [10.8.3.5.1]**
- Beta method (β-method) for cohesionless soil, as specified in **LRFD [10.8.3.5.2]**

The ultimate unit point resistance of a drilled shaft in intact or tightly jointed rock is computed as 2.5 times the unconfined compressive strength of the rock. For rock containing open or filled joints, the geomechanics RMR system is used to characterize the rock, and the ultimate point resistance in rock can be computed as specified in **LRFD [10.8.3.5.4c]**.

11.3.2.3.3 Group Capacity

For drilled shaft groups bearing in cohesive soils or ending in a strong layer overlying a weaker layer, the axial resistance is determined using the same approach as used for driven piles. For drilled shaft groups in cohesionless soil, a group efficiency factor is applied to the ultimate resistance of a single drilled shaft. The group efficiency factor is a function of the center-to-center shaft spacing and is linearly interpolated between a value of 0.65 at a center-to-center spacing of 2.5 shaft diameters and a value of 1.0 at a center-to-center spacing of 6.0 shaft diameters. This reduction is more than for driven piles at similar spacing, because construction of drilled shafts tends to loosen the soil between the shafts rather than densify it as with driven piles.

11.3.2.4 Lateral Load Resistance

Because drilled shafts are made of reinforced concrete, the lateral analysis should consider the nonlinear variation of bending stiffness with respect to applied bending moment. At small applied moments, the reinforced concrete section performs elastically based on the size of the section and the modulus of elasticity of the concrete. At larger moments, the concrete cracks in tension and the stiffness drops significantly.

11.3.2.5 Other Considerations

Detailing of the reinforcing steel in a drilled shaft must consider the constructability of the shaft. The reinforcing cages must be stiff enough to resist bending during handling and concrete placement. In addition, the spaces between reinforcement bars must be kept large enough to permit easy flow of the concrete from the center of shaft to the outside of shaft. These two requirements will generally force the use of larger, more widely spaced longitudinal and transverse reinforcement bars than would be used in the design of an above-grade column. In addition, when using hooked bars to tie the shaft to the foundation, consideration must also be given to concrete placement requirements and temporary casing removal requirements.

11.3.3 Micropiles

11.3.3.1 General

In areas of restricted access, close proximity to settlement sensitive existing structures or difficult geology, micropiles may be considered when determining the recommended foundation type. Although typically more expensive than driven pile, constructability considerations may warrant selection of micropiles as the preferred foundation type. A micropile is constructed by drilling a borehole with drill casing, placing reinforcement and grouting the hole. Micropiles are installed by methods that cause minimal disturbance to adjacent structures, soil and the environment. They can be installed in areas with restricted access and vertical clearance. Drill casing permits installation in poor ground conditions. Micropiles are installed with the same type of equipment that is used for ground anchor and grouting projects. Micropiles can be either vertical or battered.

Micropiles are used for structural support of new structures, underpinning existing structures, scour protection and seismic retrofit at existing structures. Micropiles are also used to create a reinforced soil mass for ground stabilization.

With a micropile's smaller cross-sectional area, the pile design is more frequently governed by structural and stiffness considerations. Due to the small pile diameter, point resistance is usually disregarded for design. Steel casing for micropiles is commonly delivered in 5 to 20 foot long flush-joint threaded sections. The casing is typically 5.5 to 12 inches in diameter, with yield strength of 80 ksi. Grout is mixed neat with a water/cement ratio on the order of 0.45 and an unconfined compressive strength of 4 to 6 ksi. Grade 60, 90 and 150 single reinforcement bars are generally used with centralizers.

Grout/ground bond capacity varies directly with the method of placement and pressure used to place the grout. Common methods include grout placement under gravity head, grout placement under low pressure as temporary drill steel is removed and grout placement under high pressure using a packer and regrout tube. Some regrout tubes are equipped to allow regrouting multiple times to increase pile capacity.

11.3.3.2 Design Guidance

Micropiles shall be designed using an Allowable Stress Design approach until an LRFD approach has been developed and approved by the AASHTO Bridge Subcommittee. The design of micropiles shall be done in accordance with FHWA Publication SA-97-070, *Micropile Design and Construction Guidelines Implementation Manual*. When site-specific load tests are performed, the factor of safety can be reduced from 2.5 to 2.0 to determine the allowable axial compressive load capacity of the micropile. The reduction in factor of safety is consistent with the 2005 update to the FHWA guidelines for micropile design.

11.3.4 Augered Cast-In-Place Piles

11.3.4.1 General

Augered cast-in-place (ACIP) piles are installed by drilling a hole with a hollow stem auger. When the auger reaches a design depth (elevation) or given torque, sand-cement grout or concrete is pumped through the hollow-stem auger while the auger is withdrawn from the ground. Reinforcement steel can be placed while the grout is still fluid. A single reinforcement bar can also be installed inside the hollow stem auger before the auger is extracted. ACIP piles are installed by methods that cause minimal disturbance to adjacent structures, soil and the environment. They can also be installed in areas with restricted access and vertical clearance. Temporary casing is not required. In many situations, these foundation systems can be constructed more quickly and less expensively than other deep foundation alternatives.

ACIP piles are generally available in 12- to 36-inch diameters and typically extend to depths of 60 to 70 feet. In some cases, ACIP piles have been installed to depths of more than 100 feet. The torque capacity of the drilling equipment may limit the available penetration depth of ACIP piles, especially in stiff to hard cohesive soil. Typical Wisconsin bridge contractors do not own the necessary equipment to install this type of pile.

ACIP piles may be more economical; however, there is a greater inherent risk in their installation from the quality control standpoint. There is currently no method available to determine pile capacity during construction of ACIP piles. WisDOT does not generally use this pile type unless there are very unusual design/site requirements.

11.3.4.2 Design Guidance

In the future, the FHWA will distribute a Geotechnical Engineering Circular that will provide design and construction guidance for ACIP piles. WisDOT plans to reassess the use of ACIP piles at that time.

11.4 References

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2. American Association of State Highway and Transportation Officials. *Standard Specifications for Highway Bridges*.

3. American Association of State Highway and Transportation Officials. *AASHTO LRFD Bridge Design Specifications*.

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7. Kimmerling, R.E., R.C. Bachus, P.W. Mayne, J.A. Scheider and T.E. Zettler. *Geotechnical Engineering Circular No. 6: Shallow Foundations*. FHWA Publication SA-02-054. September 2002.

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11. Michael Baker Jr., Inc. *LRFD for Highway Bridge Substructures and Earth Retaining Structures*. FHWA Publication NHI-05-095. Revised October 2006.

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13. O'Neil, M.W. and L.C. Reese. *Drilled Shafts: Construction Procedures and Design Methods*. FHWA Publication IF-99-025. August 1999.

14. O'Neil, M.W., F.C. Townsend, K.M. Hassan, A. Butler and P.S. Chan. *Load Transfer for Drilled Shafts in Intermediate Geomaterials*. FHWA Publication RD-95-172. November 1996.

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16. Sabatini, P.J. et al. *Geotechnical Engineering Circular No. 5: Evaluation of Soil and Rock Properties*. FHWA Publication IF-02-034. April 2002.

17. Terzaghi, K. and R.B. Peck. *Soil Mechanics in Engineering Practice*, 2nd Edition. Wiley. 1967.

18. Tschebotrarioff, G.P. *Foundations, Retaining and Earth Structures*, 2nd Edition. McGraw Hill. 1973.

11.5 Design Examples

WisDOT will provide design examples.

This section will be expanded later when the design examples are available.

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13.3 Location

Piers shall be located to provide a minimum interference to flood flow. In general, place the piers parallel with the direction of flood flow. Make adequate provision for drift and ice by increasing span lengths and vertical clearances, and by selecting proper pier types. Special precautions against scour are required in unstable streambeds. Navigational clearance shall be considered when placing piers for bridges over navigable waterways. Coordination with the engineer performing the hydraulic analysis is required to ensure the design freeboard is met, the potential for scour is considered, the hydraulic opening is maintained and the flood elevations are not adversely affected upstream or downstream. Refer to Chapter 8 for further details.

In the case of railroad and highway separation structures, the spacing and location of piers and abutments is usually controlled by the minimum horizontal and vertical clearances required for the roadway or the railroad. Other factors such as utilities or environmental concerns may influence the location of the piers. Sight distance can impact the horizontal clearance required for bridges crossing roadways on horizontally curved alignments. Requirements for vertical and horizontal clearances are specified in Chapter 3. Crash wall requirements are provided on Standard for Highway Over Railroad Design Requirements.

Cost may also influence the number of piers, and therefore the number of spans, used in final design. During the planning stages, an analysis should be performed to determine the most economical configuration of span lengths versus number of piers that meet all of the bridge site criteria.

13.4 Loads on Piers

The following loads shall be considered in the design of piers. Also see 13.5 for additional guidance regarding load application.

13.4.1 Dead Loads

The dead load forces, DC and DW, acting on the piers shall include reactions from the superstructure. DC dead loads include structural components and nonstructural attachments, and DW dead loads include wearing surfaces and utilities. The pier diaphragm weight may be applied through the girders. Different load factors are applied to each of these dead load types.

For a detailed discussion of the application of dead load, refer to 17.2.4.1.

13.4.2 Live Loads

The HL-93 live load shall be used for all new bridge designs and is placed in 12'-wide design lanes. If fewer lane loads are used than what the roadway width can accommodate, the loads shall be kept within their design lanes. The design lanes shall be positioned between the curbs, ignoring shoulders and medians, to maximize the effect being considered. Refer to 17.2.4.2 for a detailed description of the HL-93 live load. For pier design, particular attention should be given to the double truck load described in 17.2.4.2.4. This condition places two trucks, spaced a minimum of 50' apart, within one design lane and will often govern the maximum vertical reaction at the pier.

WisDOT policy items:

A 10 foot design lane width may be used for the distribution of live loads to a pier cap.

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

For girder type superstructures, the loads are transmitted to the pier through the girders. For pier design, simple beam distribution is used to distribute the live loads to the girders. The wheel and lane loads are therefore transversely distributed to the girders by the lever rule as opposed to the Distribution Factor Method specified in **LRFD [4.6.2.2.2]**. The lever rule linearly distributes a portion of the wheel load to a particular girder based upon the girder spacing and the distance from the girder to the wheel load. The skew of the structure is not considered when calculating these girder reactions. Refer to 17.2.10 for additional information about live load distribution to the substructure and to Figure 17.2-17 for application of the lever rule.

For slab type superstructures, the loads are assumed to be transmitted directly to the pier without any transverse distribution. This assumption is used even if the pier cap is not integral with the superstructure. The HL-93 live load is applied as concentrated wheel loads combined with a uniform lane load. The skew of the structure is considered when applying these loads to the cap. The lane width is then divided by the cosine of the skew angle, and the load is distributed over the new lane width along the pier centerline.

As a reminder, the live load force to the pier for a continuous bridge is based on the *reaction*, not the sum of the adjacent span shear values. A pier beneath non-continuous spans (at an expansion joint) uses the sum of the reactions from the adjacent spans.

13.4.3 Vehicular Braking Force

Vehicular braking force, BR, is specified in **LRFD [3.6.4]** and is taken as the greater of:

- 25% of the axle loads of the design truck
- 25% of the axle loads of the design tandem
- 5% of the design truck plus lane load
- 5% of the design tandem plus lane load

The loads applied are based on loading one-half the adjacent spans. Do not use a percentage of the live load reaction. All piers receive this load. It is assumed that the braking force will be less than the dead load times the bearing friction value and all force will be transmitted to the given pier. The tandem load, even though weighing less than the design truck, must be considered for shorter spans since not all of the axles of the design truck may be able to fit on the tributary bridge length.

This force represents the forces induced by vehicles braking and may act in all design lanes. The braking force shall assume that traffic is traveling in the same direction for all design lanes as the existing lanes may become unidirectional in the future. This force acts 6' above the bridge deck, but the longitudinal component shall be applied at the bearings. It is not possible to transfer the bending moment of the longitudinal component acting above the bearings on typical bridge structures. The multiple presence factors given by **LRFD [3.6.1.1.2]** shall be considered. Per **LRFD [3.6.2.1]**, the dynamic load allowance shall not be considered when calculating the vehicular braking force.

13.4.4 Wind Loads

WisDOT exception to AASHTO:

The design wind velocity, V_{DZ}, from **LRFD [3.8.1.1]** shall be set to 100 mph for all bridge elevations.

In 13.4.4.1 and 13.4.4.2, the base wind pressure, P_B, will not be modified based on the elevation of the bridge and shall be taken as:

$$P_D = P_B$$

Where:

P_D = Design wind pressure at all elevations (ksf)

Wind loads are divided into the following four types.

13.4.4.1 Wind Load on Superstructure

To determine WS, the base wind pressures, P_B, presented in Table 13.4-1 shall be applied to the superstructure elements as specified in **LRFD [3.8.1.2.2]**.

Wind Skew Angle (deg.)	Trusses, Columns and Arches		Girders	
	Lateral Load (ksf)	Longitudinal Load (ksf)	Lateral Load (ksf)	Longitudinal Load (ksf)
0	0.075	0.000	0.050	0.000
15	0.070	0.012	0.044	0.006
30	0.065	0.028	0.041	0.012
45	0.047	0.041	0.033	0.016
60	0.024	0.050	0.017	0.019

Table 13.4-1
Superstructure Base Wind Pressures

The wind skew angle shall be taken as measured from a perpendicular to the longitudinal axis. The wind direction used shall be that which produces the maximum force effects on the member. Transverse and longitudinal pressures shall be applied simultaneously. The longitudinal component shall be applied at the bearing elevation, and the transverse component shall be applied at its actual elevation.

WisDOT policy item:

The following conservative values for wind on superstructure, WS, may be used for all girder bridges:

- 0.05 ksf, transverse
- 0.012 ksf, longitudinal

Both forces shall be applied simultaneously. Do not apply to open rails or fences. Do apply this force to all parapets, including parapets located between the roadway and sidewalk if there is an open rail or fence on the edge of the sidewalk.

13.4.4.2 Wind Load Applied Directly to Substructure

To determine WS for wind applied directly to substructures, the base wind pressure, P_B, to be applied to the substructure units is 0.040 ksf as specified in **LRFD [3.8.1.2.3]**. This load can be resolved into components based on skew, or the following policy item can be followed:

WisDOT policy item:

The following values for wind applied directly to substructures, WS, may be used for all bridges:

- 0.040 ksf, transverse (along axis of substructure unit)
- 0.040 ksf, longitudinal (normal to axis of substructure unit)

Both forces shall be applied simultaneously.

13.4.4.3 Wind Load on Vehicles

As specified in **LRFD [3.8.1.3]** the wind force on vehicles, WL, is applied 6 ft. above the roadway. The longitudinal component shall be applied at the bearing elevation, and the transverse component shall be applied at its actual elevation.

WisDOT policy item:

The following values for wind on live load, WL, may be used for all bridges:

- 0.100 klf, transverse
- 0.040 klf, longitudinal

Both forces shall be applied simultaneously.

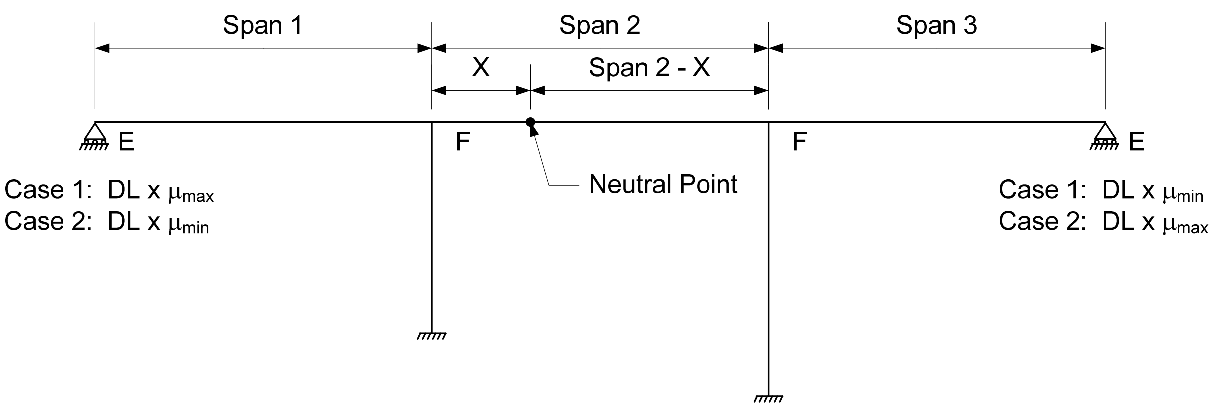
13.4.4.4 Vertical Wind Load

As specified in **LRFD [3.8.2]** an overturning vertical wind force, WS, shall be applied to limit states that do not involve wind on live load. A vertical upward wind force of 0.020 ksf times the out-to-out width of the bridge deck shall be considered a longitudinal line load. This lineal force shall be applied at the windward ¼ point of the deck, which causes the largest upward force at the windward fascia girder.

13.4.5 Uniform Temperature Forces

Temperature changes in the superstructure cause it to expand and contract along its longitudinal axis. These length changes induce forces in the substructure units based upon the fixity of the bearings, as well as the location and number of substructure units. The skew angle of the pier shall be considered when determining the temperature force components.

In determining the temperature forces, TU, applied to each substructure unit, the entire bridge superstructure length between expansion joints is considered. In all cases, there is a neutral point on the superstructure which does not move due to temperature changes. All temperature movements will then emanate outwards or inwards from this neutral point. This point is determined by assuming a neutral point. The sum of the expansion forces and fixed pier forces on one side of the assumed neutral point is then equated to the sum of the expansion forces and fixed pier forces on the other side of the assumed neutral point. Maximum friction coefficients are assumed for expansion bearings on one side of the assumed neutral point and minimum coefficients are assumed on the other side to produce the greatest unbalanced force for the fixed pier(s) on one side of the assumed neutral point. The maximum and minimum coefficients are then reversed to produce the greatest unbalanced force for the pier(s) on the other side of the assumed neutral point. For semi-expansion abutments, the assumed minimum friction coefficient is 0.06 and the maximum is 0.10. For laminated elastomeric bearings, the force transmitted to the pier is the shear force generated in the bearing due to temperature movement. Example E27-1.8 illustrates the calculation of this force. Other expansion bearing values can be found in Chapter 27 – Bearings. When writing the equation to balance forces, one can set the distance from the fixed pier immediately to one side of the assumed neutral point as 'X' and the fixed pier immediately to the other side as (Span Length – 'X'). This is illustrated in Figure 13.4-1.



$$\text{Case1: } \sum DL\, x\, \mu_{max} + \sum \frac{3EI\alpha TX}{144h^3} = \sum \frac{3EI\alpha T(\text{Span}\,2 - X)}{144h^3} + \sum DL\, x\, \mu_{min}$$

(Lt. Abutment) (Pier 1) (Pier 2) (Rt. Abutment)

Figure 13.4-1
Neutral Point Location with Multiple Fixed Piers

As used in Figure 13.4-1:

E = Column or shaft modulus of elasticity (ksi)

I	=	Column or shaft moment of inertia about longitudinal axis of the pier (in^4)
α	=	Superstructure coefficient of thermal expansion (ft/ft/°F)
T	=	Temperature change of superstructure (°F)
μ	=	Coefficient of friction of the expansion bearing (dimensionless)
h	=	Column height (ft)
DL	=	Total girder dead load reaction at the bearing (kips)
X	=	Distance between the fixed pier and the neutral point (ft)

The temperature force on a single fixed pier in a bridge is the resultant of the unbalanced forces acting on the substructure units. Maximum friction coefficients are assumed for expansion bearings on one side of the pier and minimum coefficients are assumed on the other side to produce the greatest unbalanced force on the fixed pier.

The temperature changes in superstructure length are assumed to be along the longitudinal axis of the superstructure regardless of the substructure skew angle. This assumption is more valid for steel structures than for concrete structures.

The force on a column with a fixed bearing due to a temperature change in length of the superstructure is:

$$F = \frac{3EI\alpha TL}{144h^3}$$

Where:

L	=	Superstructure expansion length between neutral point and location being considered (ft)
F	=	Force per column applied at the bearing elevation (kips)

This force shall be resolved into components along both the longitudinal and transverse axes of the pier.

The values for computing temperature forces in Table 13.4-2 shall be used on Wisconsin bridges. Do not confuse this temperature change with the temperature range used for expansion joint design.

	Reinforced Concrete	Steel
Temperature Change	45 °F	90 °F
Coefficient of Thermal Expansion	0.0000060/°F	0.0000065/°F

Table 13.4-2
Temperature Expansion Values

Temperature forces on bridges with two or more fixed piers are based on the movement of the superstructure along its centerline. These forces are assumed to act normal and parallel to the longitudinal axis of the pier as resolved through the skew angle. The lateral restraint offered by the superstructure is usually ignored. Except in unusual cases, the larger stiffness generated by considering the transverse stiffness of skewed piers is ignored.

13.4.6 Force of Stream Current

The force of flowing water, WA, acting on piers is specified in **LRFD [3.7.3]**. This force acts in both the longitudinal and transverse directions.

13.4.6.1 Longitudinal Force

The longitudinal force is computed as follows:

$$p = \frac{C_D V^2}{1{,}000}$$

Where:

p = Pressure of flowing water (ksf)

V = Water design velocity for the design flood in strength and service limit states and for the check flood in the extreme event limit state (ft/sec)

C_D = Drag coefficient for piers (dimensionless), equal to 0.7 for semicircular-nosed piers, 1.4 for square-ended piers, 1.4 for debris lodged against the pier and 0.8 for wedged-nosed piers with nose angle of 90° or less

The longitudinal drag force shall be computed as the product of the longitudinal stream pressure and the projected exposed pier area.

13.4.6.2 Lateral Force

The lateral force is computed as follows:

$$p = \frac{C_D V^2}{1{,}000}$$

Where:

p = Lateral pressure of flowing water (ksf)

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

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

Table 13.4-3
Lateral Drag Coefficient Values

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

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

13.4.7 Buoyancy

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

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

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

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

Table 13.4-4
Submerged Unit Weights of Various Soils

13.4.8 Ice

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

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

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

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

13.4.8.1 Force of Floating Ice and Drift

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

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

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

$$F = F_c = C_a ptw$$

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

Where:

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

WisDOT policy item:

Since the angle of inclination of the pier nose with respect to the vertical is always less than or equal to 15° on standard piers in Wisconsin, the flexural ice failure mode does not need to be considered for these standard piers ($f_b = 0$).

WisDOT policy item:

If the pier is approximately aligned with the direction of the ice flow, only the first design case as specified in **LRFD [3.9.2.4]** shall be investigated due to the unknowns associated with the friction angle defined in the second design case.

A longitudinal force equal to F shall be combined with a transverse force of 0.15F

Both the longitudinal and transverse forces act simultaneously at the pier nose.

If the pier is located such that its longitudinal axis is skewed to the direction of the ice flow, the ice force on the pier shall be applied to the projected pier width and resolved into components. In this condition, the transverse force to the longitudinal axis of the pier shall be a minimum of 20% of the total force.

WisDOT exception to AASHTO:

Based upon the pier geometry in the Standards, the ice loadings of **LRFD [3.9.4]** and **LRFD [3.9.5]** shall be ignored.

13.4.8.2 Force Exerted by Expanding Ice Sheet

Expansion of an ice sheet, resulting from a temperature rise after a cold wave, can develop considerable force against abutting structures. This force can result if the sheet is restrained between two adjacent bridge piers or between a bluff type shore and bridge pier. The force direction is therefore transverse to the direction of stream flow.

Force from ice sheets depends upon ice thickness, maximum rate of air-temperature rise, extent of restraint of ice and extent of exposure to solar radiation. In the absence of more precise information, estimate an ice thickness and use a force of 8.0 ksf.

It is not necessary to design all bridge piers for expanding ice. If one side of a pier is exposed to sunlight and the other side is in the shade along with the shore in the pier vicinity, consider the development of pressure from expanding ice. If the central part of the ice is exposed to the sun's radiation, consider the effect of solar energy, which causes the ice to expand.

13.4.9 Centrifugal Force

Centrifugal force, CE, is specified in **LRFD [3.6.3]** and is included in the pier design for structures on horizontal curves. The lane load portion of the HL-93 loading is neglected in the computation of the centrifugal force.

The centrifugal force is taken as the product of the axle weights of the design truck or tandem and the factor, C, given by the following equation:

$$C = \frac{4}{3}\frac{v^2}{gR}$$

Where:

V = Highway design speed (ft/sec)

g = Gravitational acceleration = 32.2 (ft/sec^2)

R = Radius of curvature of travel lane (ft)

The multiple presence factors specified in **LRFD [3.6.1.1.2]** shall apply to centrifugal force.

Centrifugal force is assumed to act radially and horizontally 6’ above the roadway surface. The point 6’ above the roadway surface is measured from the centerline of roadway. The design speed may be determined from the Wisconsin *Facilities Development Manual*, Chapter 11. It is not necessary to consider the effect of superelevation when centrifugal force is used, because the centrifugal force application point considers superelevation.

13.4.10 Extreme Event Collision Loads

WisDOT exception to AASHTO:

LRFD [3.6.5] for vehicular collision force, CT, shall be followed as stated except:

LRFD [3.6.5.1] and **LRFD [3.6.5.2]** shall be considered equal alternatives, meaning that protecting the pier and designing the pier for the 400 kip static force are each equally acceptable. The bridge design engineer should work with the roadway engineer to determine which alternative is preferred.

WisDOT policy item:

Unless protected by a TL-5 barrier, embankment or adequate offset, the pier columns/shaft, *only*, shall be strengthened to comply with **LRFD [3.6.5]**. For a multi-column pier the minimum size column shall be 3x5 ft rectangular or 4 ft diameter (consider clearance issues and/or the wide cap required when using 4 ft diameter columns). Hammerhead pier shafts are considered adequately sized.

The vertical reinforcement for the columns/shaft shall be the greater of what is required by design (not including the Extreme Event II loading) or a minimum of 1.5% of the gross concrete section (total cross section without deduction for rustications less than or equal to 1-1/2" deep) to address the collision force for the 3x5 ft rectangular and 4 ft diameter columns. The 1.5% minimum for 15 sq. ft. may be prorated down to 1% minimum for sections with at least a 30 sq. ft. cross sectional area.

For the 3x5 ft rectangular columns, use double #5 stirrups spaced at 6" vertically as a minimum. For the 4 ft diameter columns, use #4 spiral reinforcement (smooth bars) spaced vertically at 6" as a minimum. Hammerhead pier shafts shall have, as a minimum, the horizontal reinforcement as shown on the Standards.

WisDOT exception to AASHTO:

The vessel collision load, CV, in **LRFD [3.14]** will not be applied to every navigable waterway of depths greater than 2'. For piers located in navigable waterways, the engineer shall contact the WisDOT project manager to determine if a vessel collision load is applicable.

13.5 Load Application

When determining pier design forces, a thorough understanding of the load paths for each load is critical to arriving at loads that are reasonable per *AASHTO LRFD*. The assumptions associated with different pier, bearing and superstructure configurations are also important to understand. This section provides general guidelines for the application of forces to typical highway bridge piers.

13.5.1 Loading Combinations

Piers are designed for the Strength I, Strength III, Strength V and Extreme Event II load combinations as specified in **LRFD [3.4.1]**. Reinforced concrete pier components are also checked for the Service I load combination. Load factors for these load combinations are presented in Table 13.5-1. See 13.10 for loads applicable to pile bents and pile encased piers.

Load Combination Limit State	Load Factor										
	DC		DW		LL+IM BR CE	WA	WS	WL	FR	TU CR SH	IC CT CV
	Max.	Min.	Max.	Min.							
Strength I	1.25	0.90	1.50	0.65	1.75	1.00	0.00	0.00	1.00	0.50	0.00
Strength III	1.25	0.90	1.50	0.65	0.00	1.00	1.40	0.00	1.00	0.50	0.00
Strength V	1.25	0.90	1.50	0.65	1.35	1.00	0.40	1.00	1.00	0.50	0.00
Service I	1.00	1.00	1.00	1.00	1.00	1.00	0.30	1.00	1.00	1.00	0.00
Extreme Event II	1.25	0.90	1.50	0.65	0.50	1.00	0.00	0.00	1.00	0.00	1.00

Table 13.5-1
Load Factors

13.5.2 Expansion Piers

See 13.4 for additional guidance regarding the application of specific loads.

Transverse forces applied to expansion piers from the superstructure include loads from one-half of the adjacent span lengths, and are applied at the location of the transverse load.

For expansion bearings other than elastomeric, longitudinal forces are transmitted to expansion piers through friction in the bearings. These forces, other than temperature, are based on loading one-half of the adjacent span lengths, with the maximum being no greater than the maximum friction force (dead load times the maximum friction coefficient of a sliding bearing). See 27.2.2 to determine the bearing friction coefficient. The longitudinal forces are applied at the bearing elevation.

Expansion piers with elastomeric bearings are designed based on the force that the bearings resist, with longitudinal force being applied at the bearing elevation. This force is applied as some combination of temperature force, braking force, and/or wind load depending on what load case generates the largest deflection at the bearing. The magnitude of the force shall be computed as follows:

$$F = \frac{GA\Delta n}{t}$$

Where:

F	=	Elastomeric bearing force used for pier design (kips)
G	=	Shear modulus of the elastomer (ksi)
A	=	Bearing pad area (in^2)
Δ	=	Deflection at bearing from thermal or braking force (in)
n	=	Number of bearings per girder line; typically one for continuous steel girders and two for prestressed concrete beams (dimensionless)
t	=	Total elastomer thickness (without steel laminates) (in)

Example E27-1.8 illustrates the calculation of this force.

See 13.4.5 for a discussion and example of temperature force application for all piers.

13.5.3 Fixed Piers

Transverse forces applied to expansion piers from the superstructure include loads from one-half of the adjacent span lengths, and are applied at the location of the transverse load. For fixed bearings, longitudinal forces, other than temperature, are based on loading one-half of the adjacent span lengths. The longitudinal forces are applied at the bearing elevation.

See 13.4.5 for a discussion and example of temperature force application for all piers.

13.6 Multi-Column Pier and Cap Design

WisDOT policy item:

Multi-column pier caps shall be designed using conventional beam theory.

The first step in the analysis of a pier frame is to determine the trial geometry of the frame components. The individual components of the frame must meet the minimum dimensions specified in 13.2.1 and as shown on the Standards. Each of the components should be sized for function, economy and aesthetics. Once a trial configuration is determined, analyze the frame and adjust the cap, columns and footings if necessary to accommodate the design loads.

When the length between the outer columns of a pier cap exceeds 65', temperature and shrinkage should be considered in the design of the columns. These effects induce moments in the columns due to the expansion and contraction of the cap combined with the rigid connection between the cap and columns. A 0.5 factor is specified in the strength limit state for the temperature and shrinkage forces to account for the long-term column cracking that occurs. A full section modulus is then used for this multi-column pier analysis. Use an increase in temperature of +35 degrees F and a decrease of -45 degrees F. Shrinkage (0.0003 ft/ft) will offset the increased temperature force. For shrinkage, the keyed vertical construction joint as required on the Standard for Multi-Columned Pier, is to be considered effective in reducing the cap length. For all temperature forces, the entire length from exterior column to exterior column shall be used.

The maximum column spacing on pier frames is 25'. Column height is determined by the bearing elevations, the bottom of footing elevation and the required footing depth. The pier cap/column and column/footing interfaces are assumed to be rigid.

The pier is analyzed as a frame bent by any of the available analysis procedures considering sidesway of the frame due to the applied loading. The gross concrete areas of the components are used to compute their moments of inertia for analysis purposes. The effect of the reinforcing steel on the moment of inertia is neglected.

Vertical loads are applied to the pier through the superstructure. The vertical loads are varied to produce the maximum moments and shears at various positions throughout the structure in combination with the horizontal forces. The effect of length changes in the cap due to temperature is also considered in computing maximum moments and shears. All these forces produce several loading conditions on the structure which must be separated to get the maximum effect at each point in the structure. The maximum moments, shears and axial forces from the analysis routines are used to design the individual pier components. Moments at the face of column are used for pier cap design.

Skin reinforcement on the side of the cap, shall be determined as per **LRFD [5.7.3.4]**. This reinforcement shall not be included in any strength calculations.

See 13.1 and 13.2.1 for further requirements specific to this pier type.

13.7.4 Design Tension Tie Reinforcement

Tension ties shall be designed to resist the strength limit state force per **LRFD [5.6.3.4.1]**. For non-prestressed caps, the tension tie steel shall satisfy:

$$A_{st} \geq \frac{P_u}{\phi f_y}$$

Where:

A_{st} = Total area of mild steel reinforcement in the tie (in^2)

P_u = Tension tie force from strength limit state (kips)

ϕ = Resistance factor for tension on reinforced concrete, equal to 0.90, as specified in **LRFD [5.5.4.2]** (dimensionless)

f_y = Yield strength of reinforcement (ksi)

Horizontal tension ties, such as ties A-B and E-F in Figure 13.7-2, are used to determine the longitudinal reinforcement required in the top of the pier cap. The maximum tension tie value should be used to calculate the top longitudinal reinforcement.

Vertical tension ties, such as ties B-H and C-I, are used to determine the vertical stirrup requirements in the cap. Similar to traditional shear design, two stirrup legs shall be accounted for when computing A_{st}. In Figure 13.7-2, the number of stirrups, n, necessary to provide the A_{st} required for tie B-H shall be spread out across Stirrup Region 2. The length limits of Stirrup Region 2 are from the midpoint between nodes A and B to the midpoint between nodes B and C. When vertical ties are located adjacent to columns, such as with tie C-I, the stirrup region extends to the column face. Therefore, the length limits of Stirrup Region 1 are from the column face to the midpoint between nodes B and C. The stirrup spacing shall then be determined by the following equation:

$$s_{max} = \frac{L}{n}$$

Where:

s_{max} = Maximum allowable stirrup spacing (in)

L = Length of stirrup region (in)

n = Number of stirrups required to satisfy the A_{st} required to resist the vertical tension tie force

Skin reinforcement on the side of the cap, shall be determined as per **LRFD [5.7.3.4]**. This reinforcement shall not be included in any strength calculations.

Check the Compression Strut Capacity

Compression struts shall be designed to resist the strength limit state force per **LRFD [5.6.3.3]**. The resistance of an unreinforced compression strut shall be taken as:

$$P_r = \phi f_{cu} A_{cs} \geq P_u$$

Where:

P_r	=	Factored resistance of compression strut (kips)
P_u	=	Compression strut force from strength limit state (kips)
ϕ	=	Resistance factor for compression in strut-and-tie models, equal to 0.70, as specified in **LRFD [5.5.4.2]** (dimensionless)
f_{cu}	=	Limiting compressive stress (ksi)
A_{cs}	=	Effective cross-sectional area of strut (in^2)

The limiting compressive stress shall be given by:

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

In which:

$$\varepsilon_1 = \varepsilon_s + (\varepsilon_s + 0.002)\cot^2 \alpha_s$$

Where:

ε_s	=	Concrete tensile strain in the direction of the tension tie at the strength limit state (in/in)
α_s	=	Smallest angle between the compression strut and the adjoining tension ties (°)
f'_c	=	Specified compressive strength (ksi)

The concrete tensile strain is given by:

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

Where:

E_s = Modulus of elasticity of steel, taken as 29,000 (ksi)

The cross-sectional area of the strut, A_{cs}, is determined by considering both the available concrete area and the anchorage conditions at the end of the strut. Figure 13.7-3, Figure 13.7-4 and Figure 13.7-5 illustrate the computation of A_{cs}.

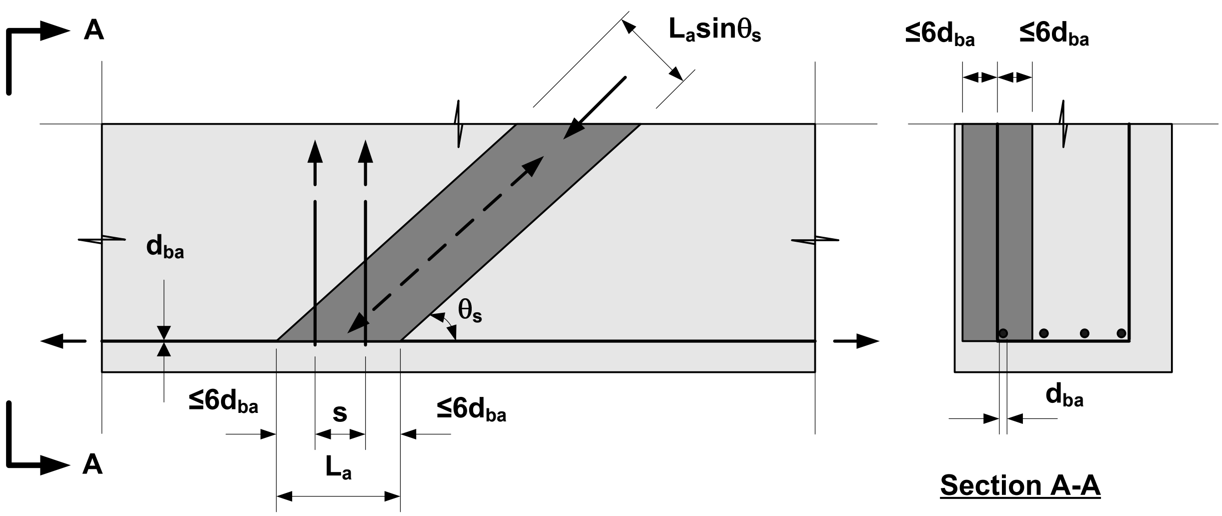


Figure 13.7-3
Strut Anchored by Tension Reinforcement Only

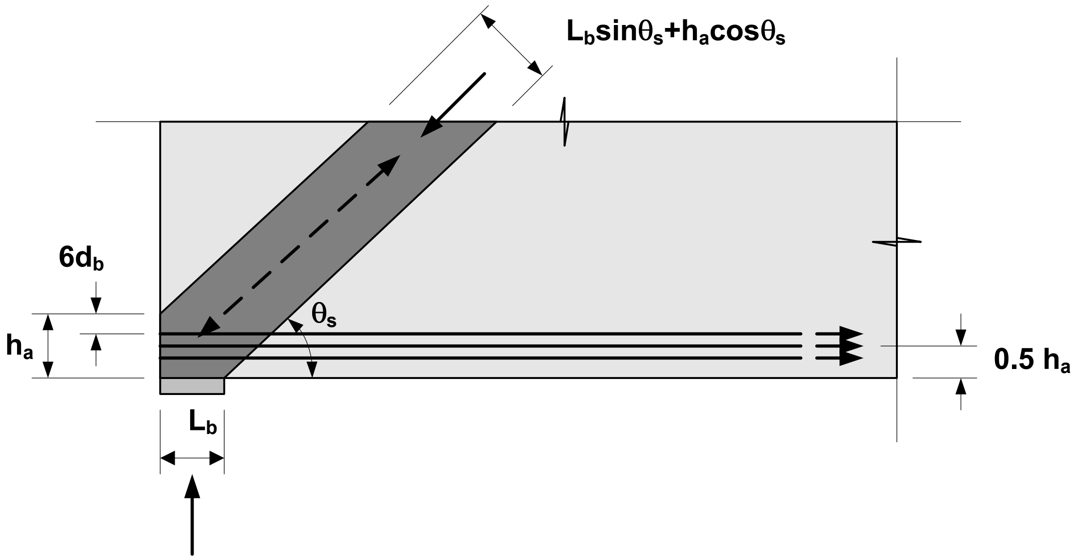


Figure 13.7-4
Strut Anchored by Bearing and Tension Reinforcement

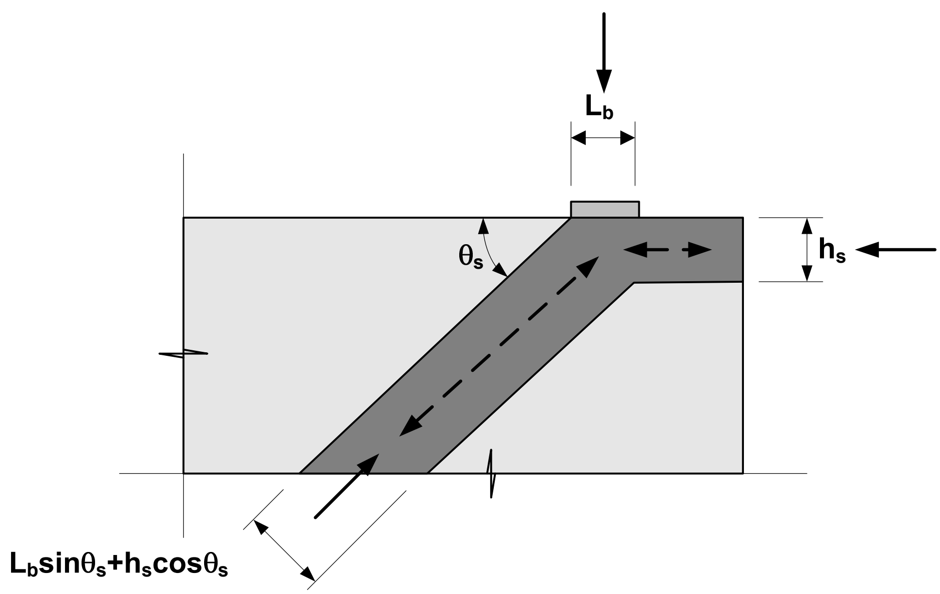


Figure 13.7-5
Strut Anchored by Bearing and Strut

Table of Contents

14.1 Introduction

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

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

14.1.1 Wall Development Process

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

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

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

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

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

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

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

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

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

14.1.1.1 Wall Numbering System

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

The wall number should be assigned in accordance with 2.5 of this manual. The only walls not requiring a number are cast-in-place concrete walls being utilized as bridge abutment wings and those walls whose height does not exceed 5.0 foot at any given point along the wall length. Wall height is measured from top of leveling pad or footing to the bottom of wall cap.

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.

Some of the 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. The walls are limited to a maximum design height of 8 feet. 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 may be segmental or full wall height 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.

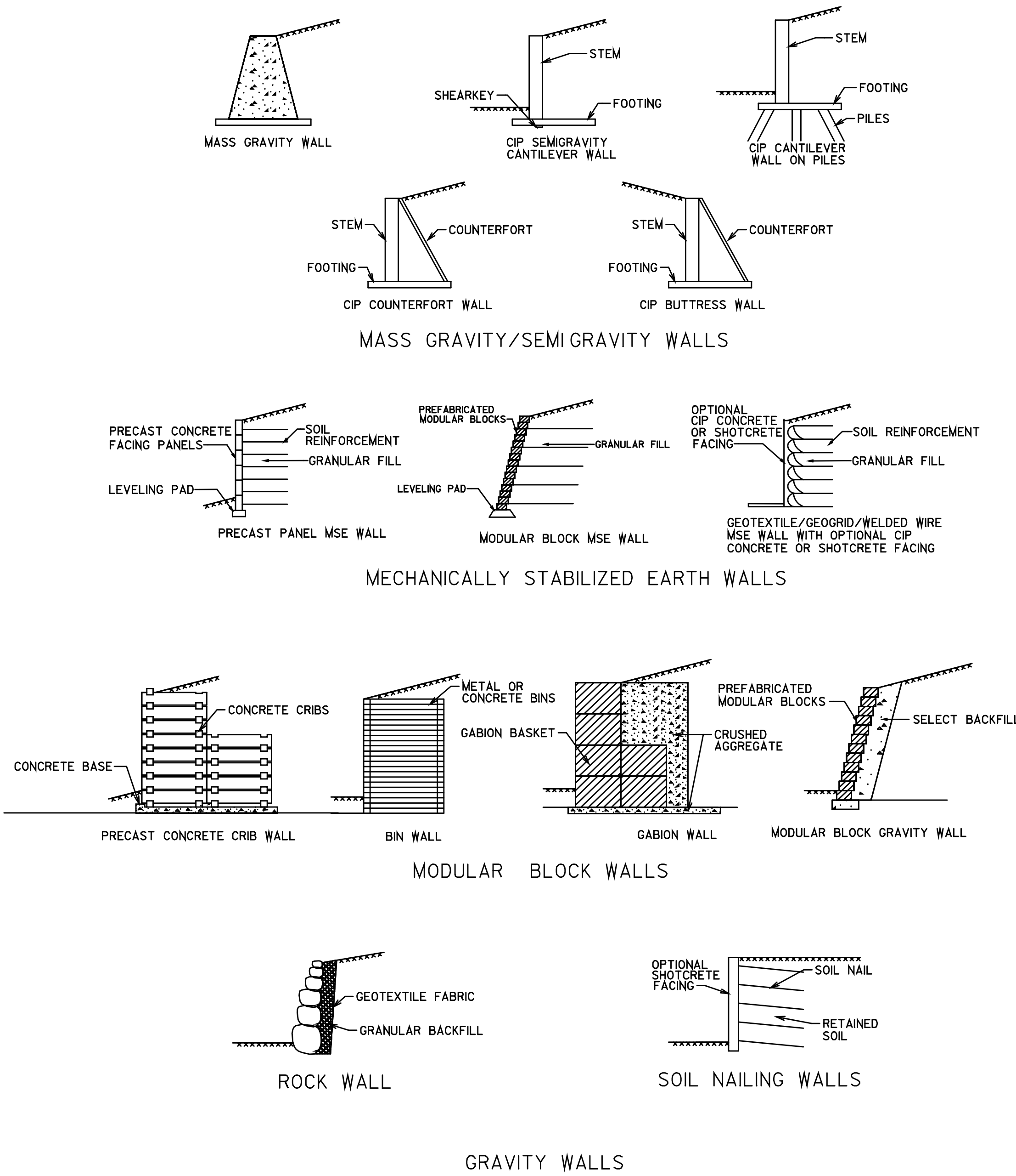


Figure 14.2-1
Gravity Walls

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.

Post and Panel Walls: Post and panel wall systems are also known as soldier pile and lagging wall systems. 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. Generally, H-piles in drilled holes are filled concrete. The soil behind the wall is retained by lagging. The lagging may be steel, wood, or concrete.

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 anchored post and panel 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 devise is generally a deadman, screw-type, or grouted tieback anchor. Anchored walls can be built to significant heights.

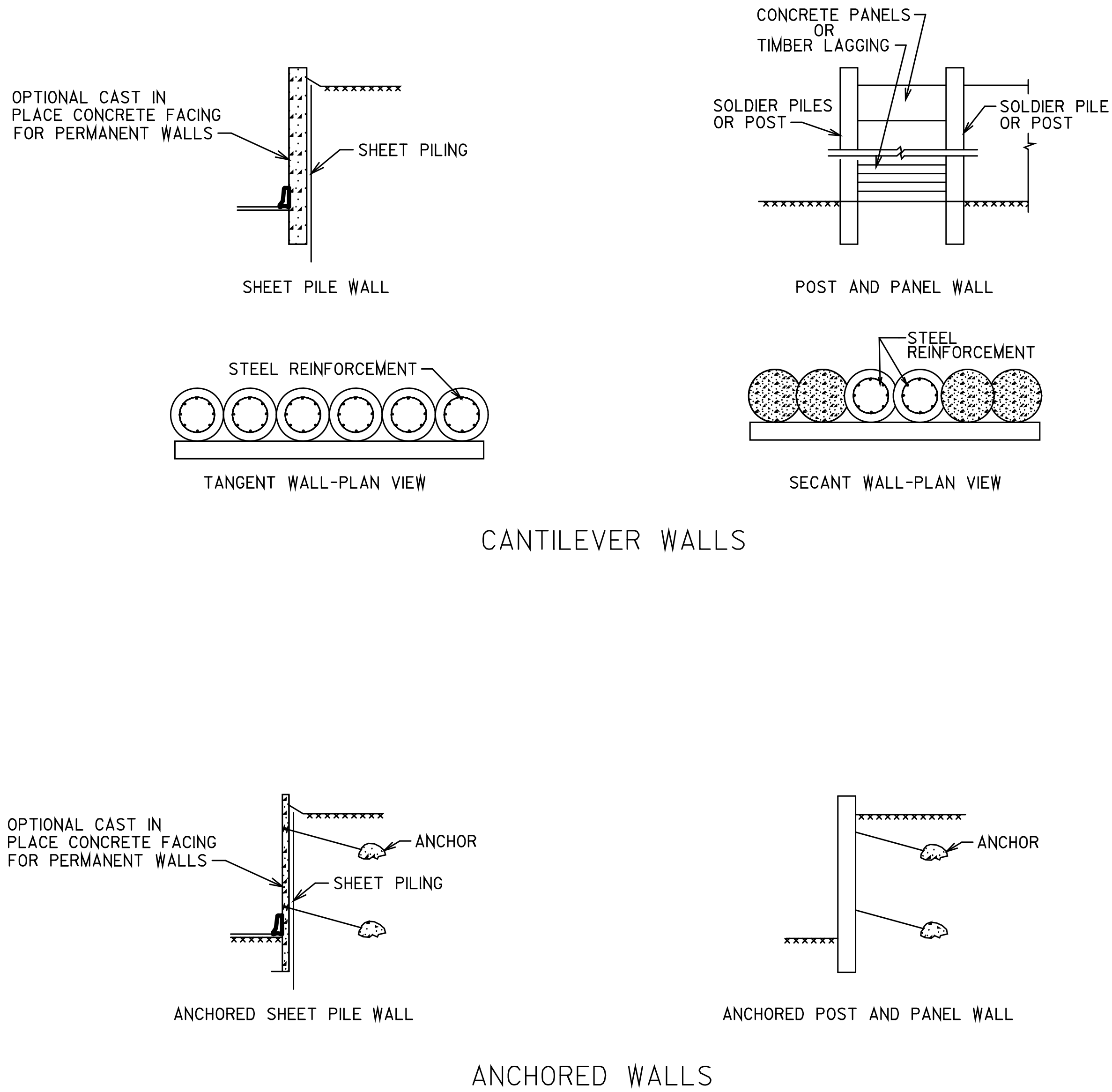


Figure 14.2-2
Non-Gravity Walls

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.

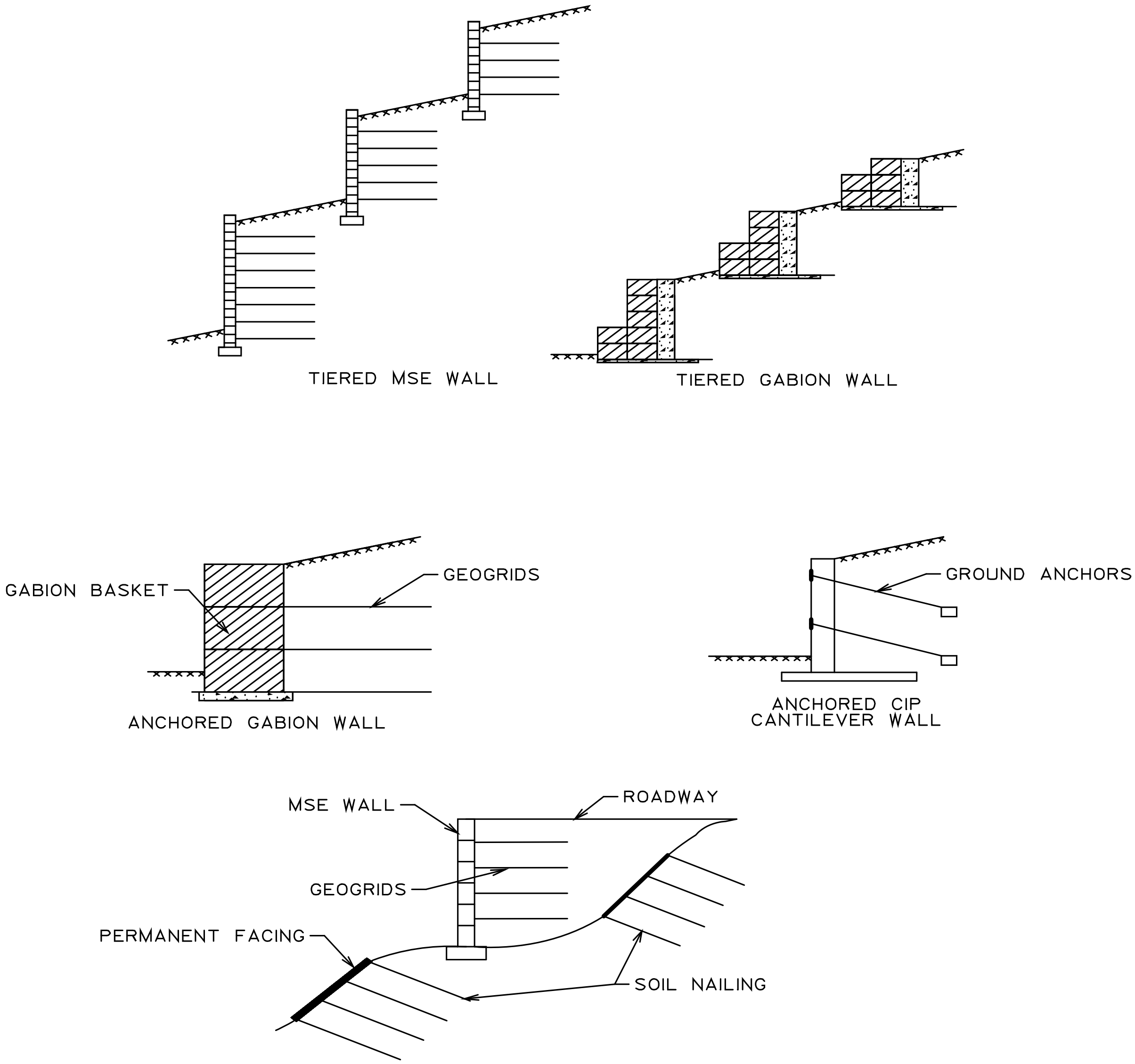


Figure 14.2-3
Tiered & Hybrid Wall Systems

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 Post & Panel Tangent/Secant	Top Down (Cut)	No
	Anchored	Anchored Sheet Pile, Post and Panel, and 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.

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)	Reqd. ROW	Advantages	Disadvantage
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 fire 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)	Reqd. ROW	Water Tightness	Advantages	Disadvantage
Sheet Pile	√	√	6-15	Minimal	fair	• Rapid construction • Readily available	• Deep foundation may be needed • Longer construction time
Post and Panel	√	√	6-28	.4H-.7H	poor	• Rapid 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		√	20-60	.4H-.7H	fair	• Adaptable to irregular layout • Can control wall stiffness	• Difficult to make height adjustment in the field • High cost
Anchored Wall	√	√	6-35	.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

The design procedures and criteria established by WisDOT are generally in conformance with *AASHTO Load and Resistance Factor Design Specifications 5*th Edition 2010 with latest interim specifications, hereafter referred as to as *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]**

$$Q = \sum \eta_i \gamma_I Q_i \le \phi R_n = R_r$$ **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.

<u>Active Earth Pressure</u>

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.

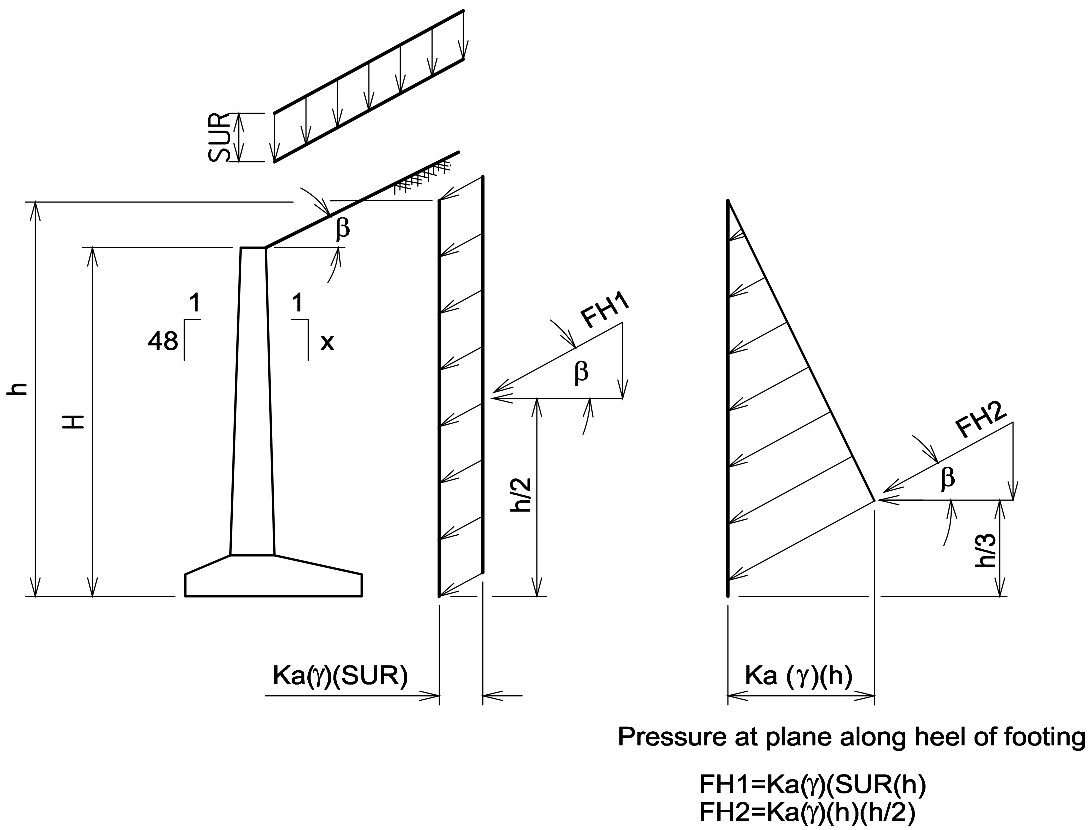


Figure14.4-1
Loading Diagram for a Cantilever Retaining Wall with Surcharge Loading

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

$K_a = \tan^2(45\text{-}\Phi_f/2)$

Where:

K_a = Coefficient of active earth pressure

Φ_f = Angle of internal friction of retained earth

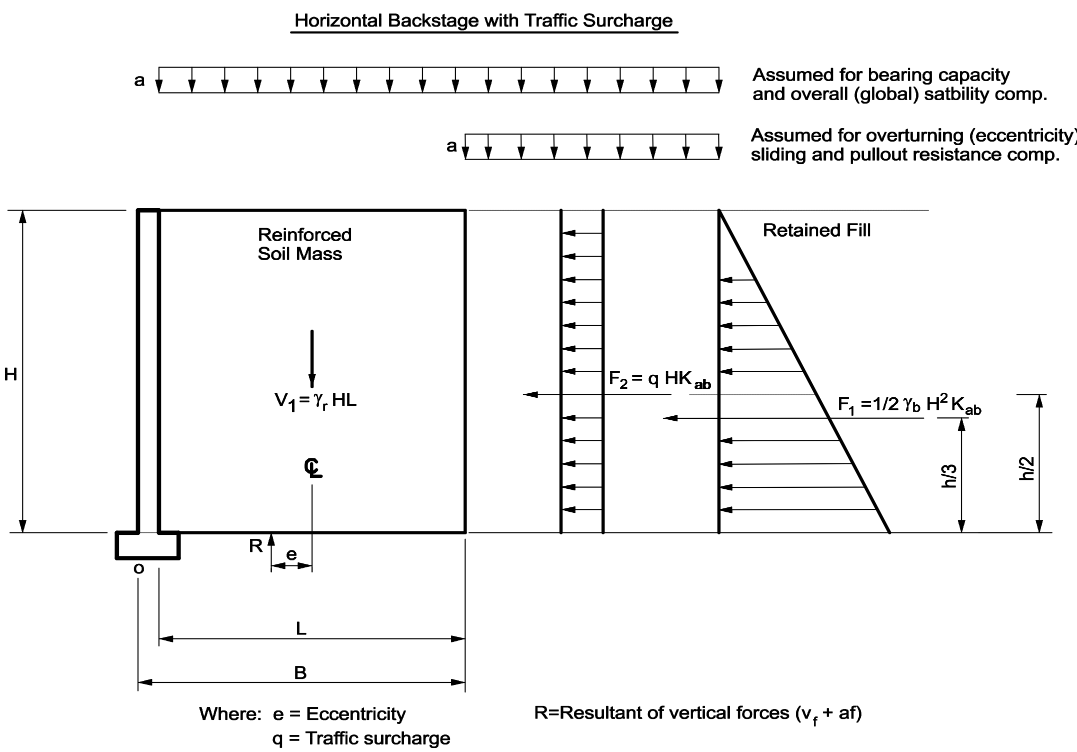


Figure 14.4-2
MSE Walls Earth Pressure for Horizontal Backslope with Traffic Surcharge
(Source *AASHTO LRFD*)

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

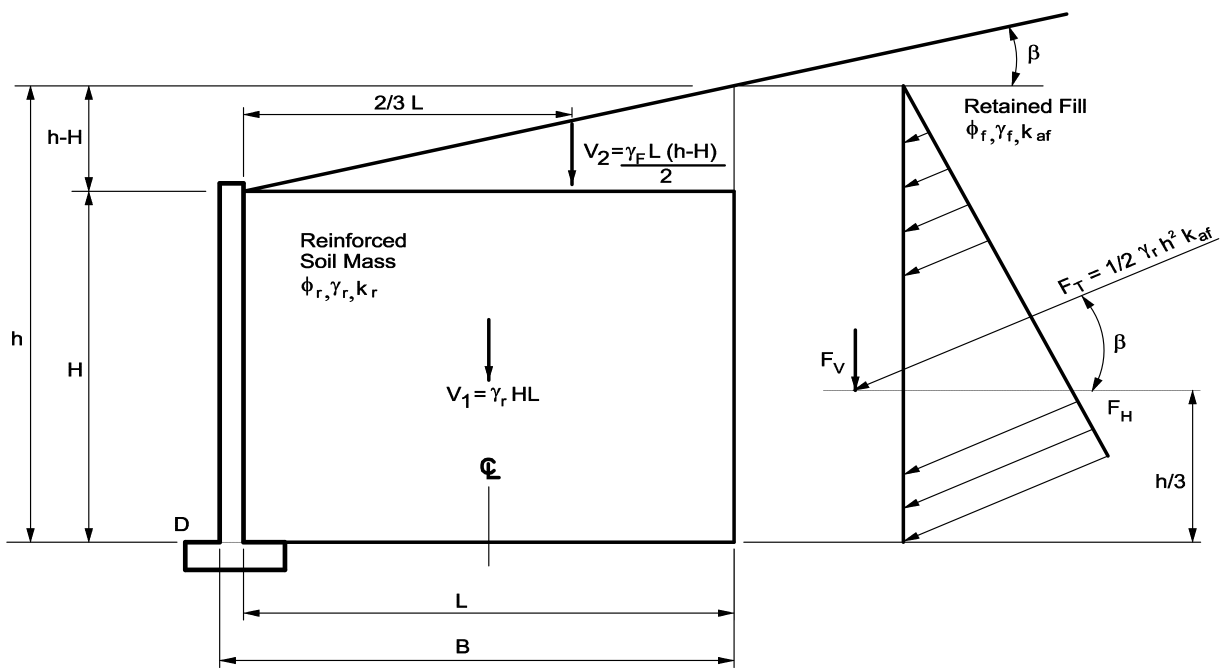


Figure 14.4-3
MSE Walls Earth Pressure for Sloping Backfill
(Source *AASHTO LRFD*)

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:

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

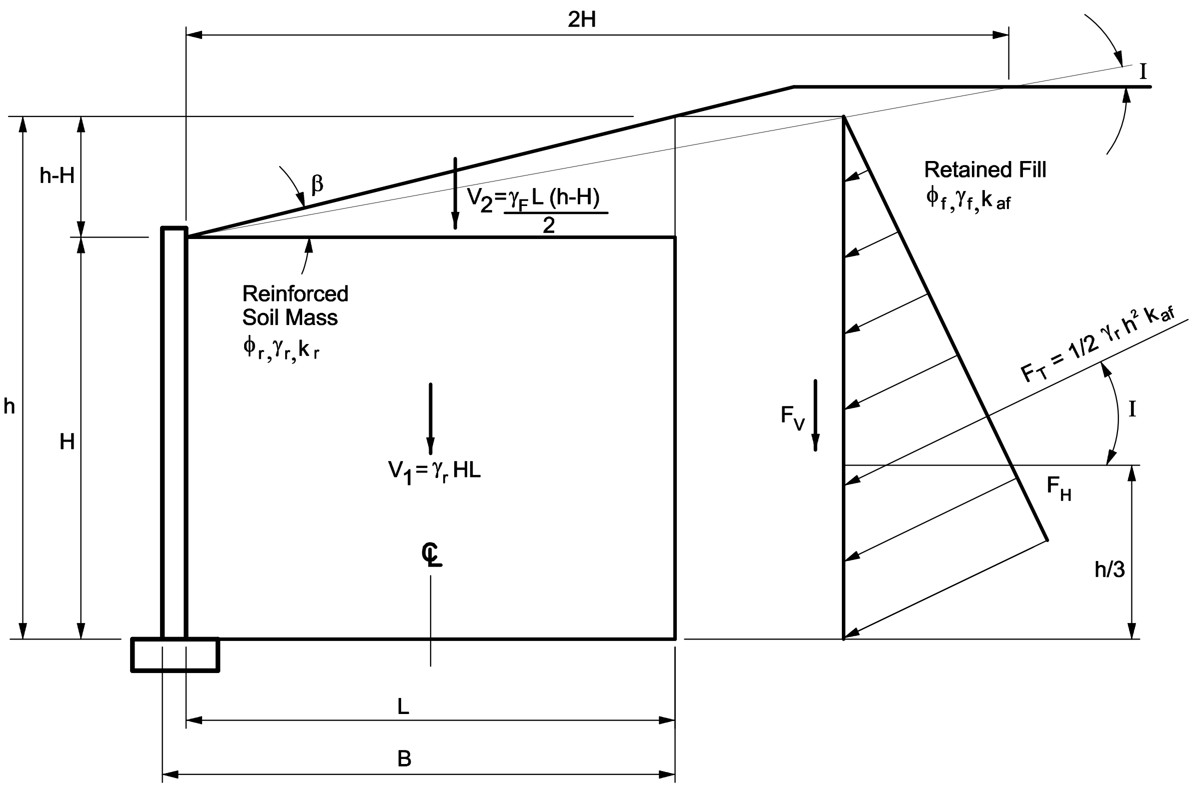


Figure 14.4-4
MSE Walls Earth Pressure for Broken Backfill
(Source *AASHTO LRFD*)

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

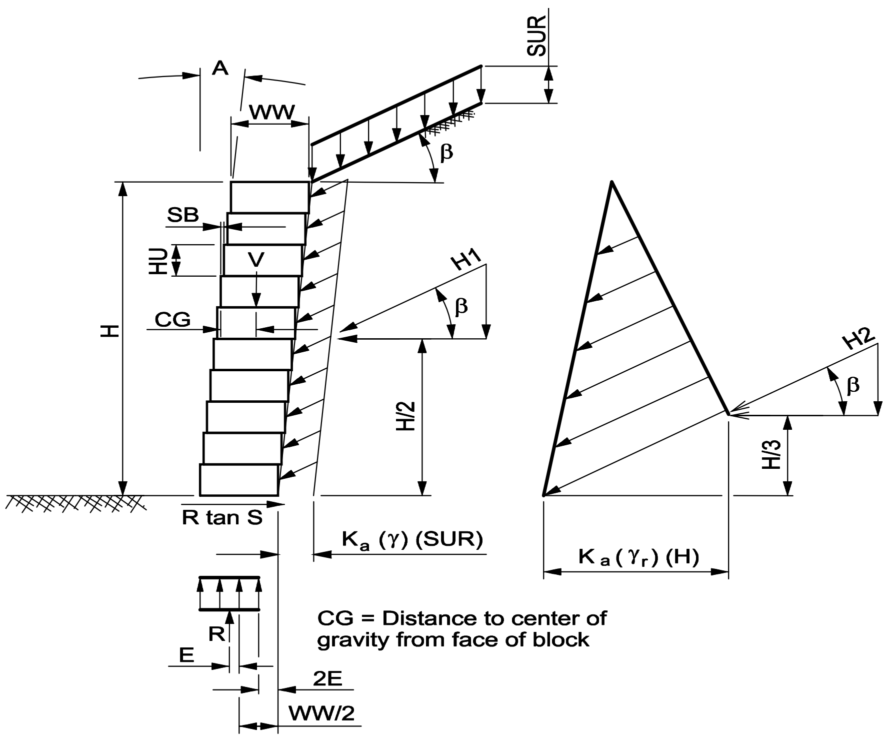


Figure 14.4-5
Modular Block Gravity Wall Analysis

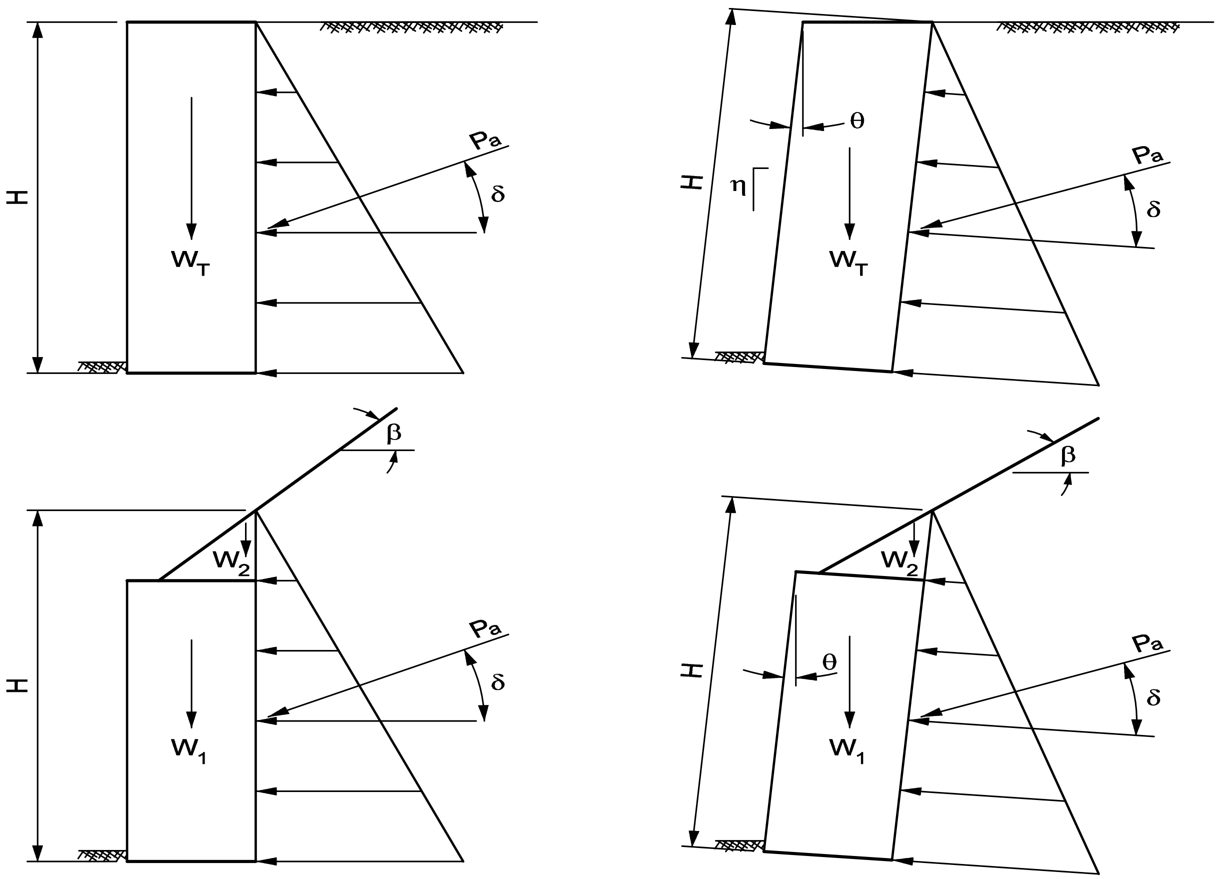
No live load traffic and live load surcharge shall be allowed on modular block gravity walls although they are designed for a minimum live load of 100psf. The density of the blocks is assumed to be 135 pcf and the drainage aggregate inside or between the blocks 120 pcf. The forces acting on a modular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

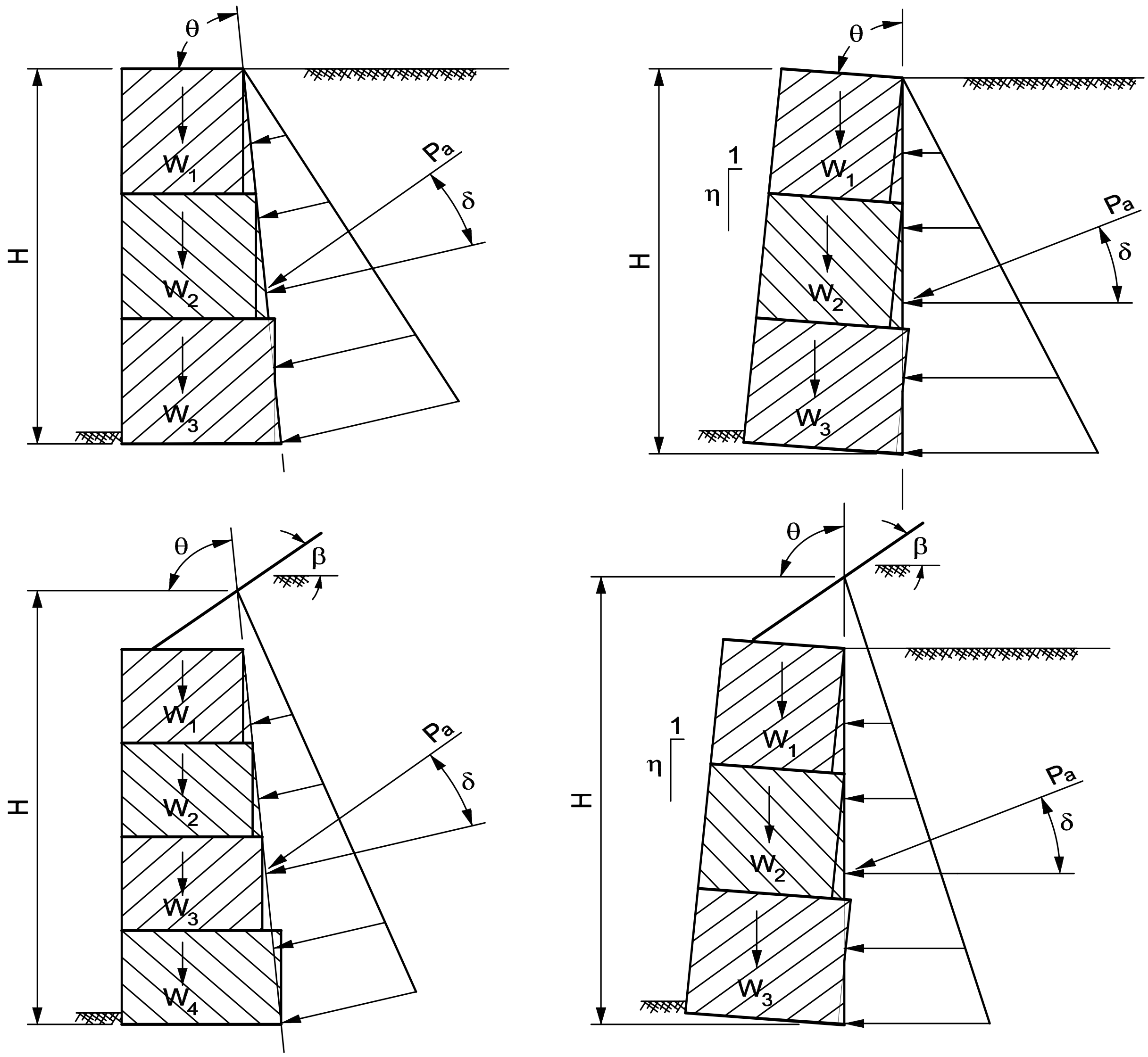


P_a @ H/3

$P_a = \frac{1}{2}\gamma H^2 K_a$

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

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*)

14.4.5.5 Load factors and Load Combinations

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

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

Table 14.4-1
Load Factors

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

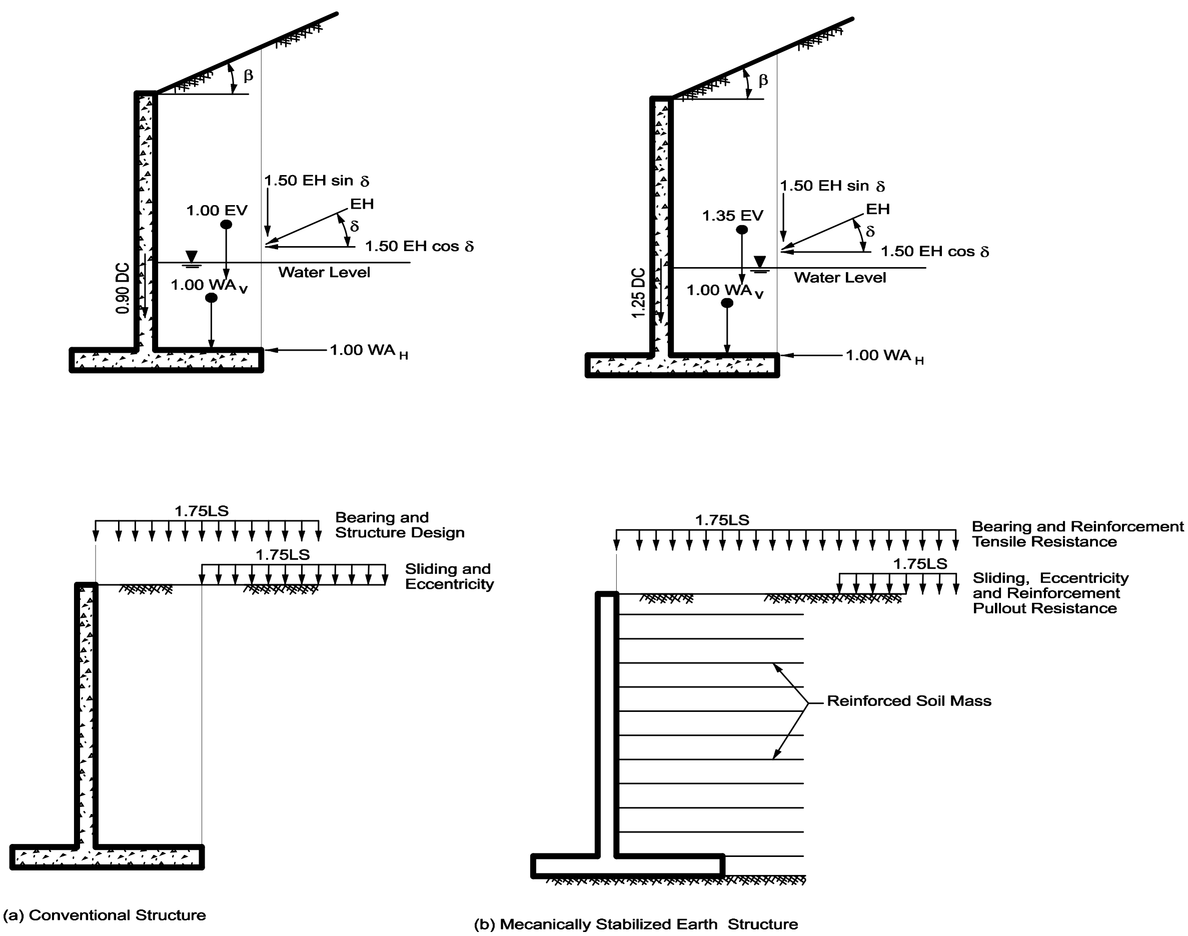


Figure 14.4-8
Application of Load Factors
(Source *AASHTO LRFD*)

14.4.5.6 Resistance Requirements and Resistance Factors

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

Where

R_r = Factored resistance

R_n = Nominal resistance recommended in the Geotechnical Report

ϕ = Resistance factor

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

14.4.6 Material Properties

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

Granular Backfill Soil Properties:

Internal Friction angle of backfill ϕ_f = 30 degrees

Backfill cohesion c = 0 psf

Unit Weight 'γ_f' = 120 pcf

Concrete:

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

Unit Weight = 150 pcf

Steel reinforcement:

Yield strength f_y = 60,000 psi

Modulus of elasticity E_s = 29,000 ksi

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

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

14.4.7 Wall Stability Checks

During the design process, walls shall be checked for anticipated failure mechanisms relating to external stability, internal stability (where applicable), movement and overall stability. In general, external and internal stability of the walls should be investigated at Strength limit states, in accordance with **LRFD [11.5.1]**. In addition, investigate the wall stability for excessive vertical and lateral displacement and overall stability at the Service limit states in accordance with **LRFD [11.5.2]**. Figure 14.4-2 thru Figure 14.4-14 show anticipated failure mechanisms for various types of walls.

14.4.7.1 External Stability

The external stability should be satisfied (generally performed by the Geotechnical Engineer) for all walls. The external stability check should include failure against lateral sliding, overturning (eccentricity), and bearing pressure failure as applicable for gravity or non-gravity wall systems in accordance with **LRFD [11.5.3]**. External stability checks should be performed at the Strength I limit state.

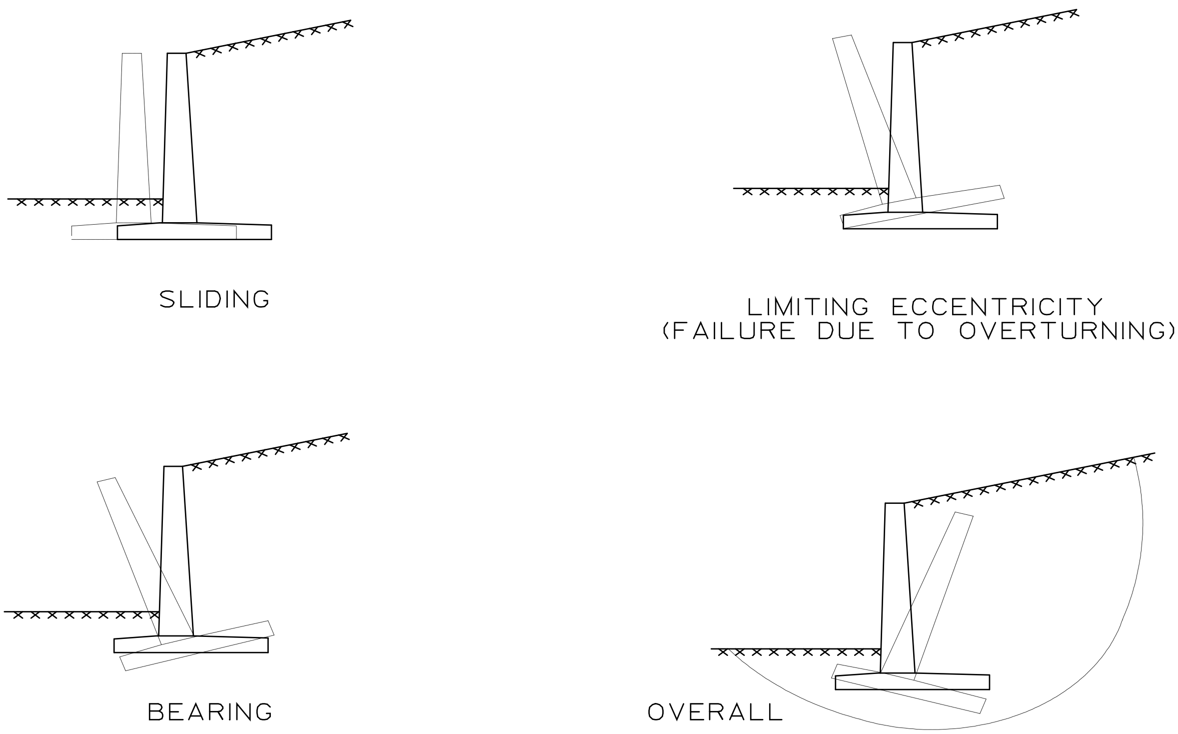


Figure14.4-9
External Stability Failure of CIP Semi-Gravity Walls

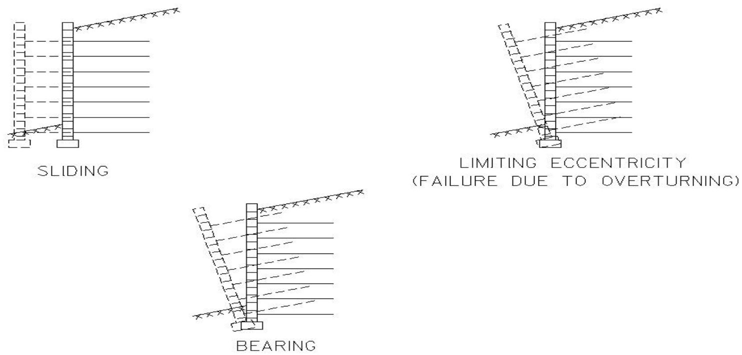


Figure 14.4-10
External Stability Failure of MSE Walls

Figure 14.4-11
Internal Stability Failure of MSE Walls

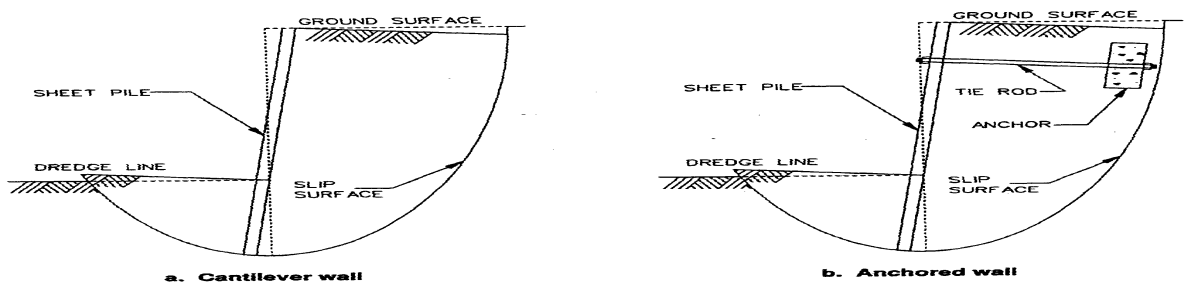


Figure 14.4-12
Deep Seated Failure of Non-Gravity Walls

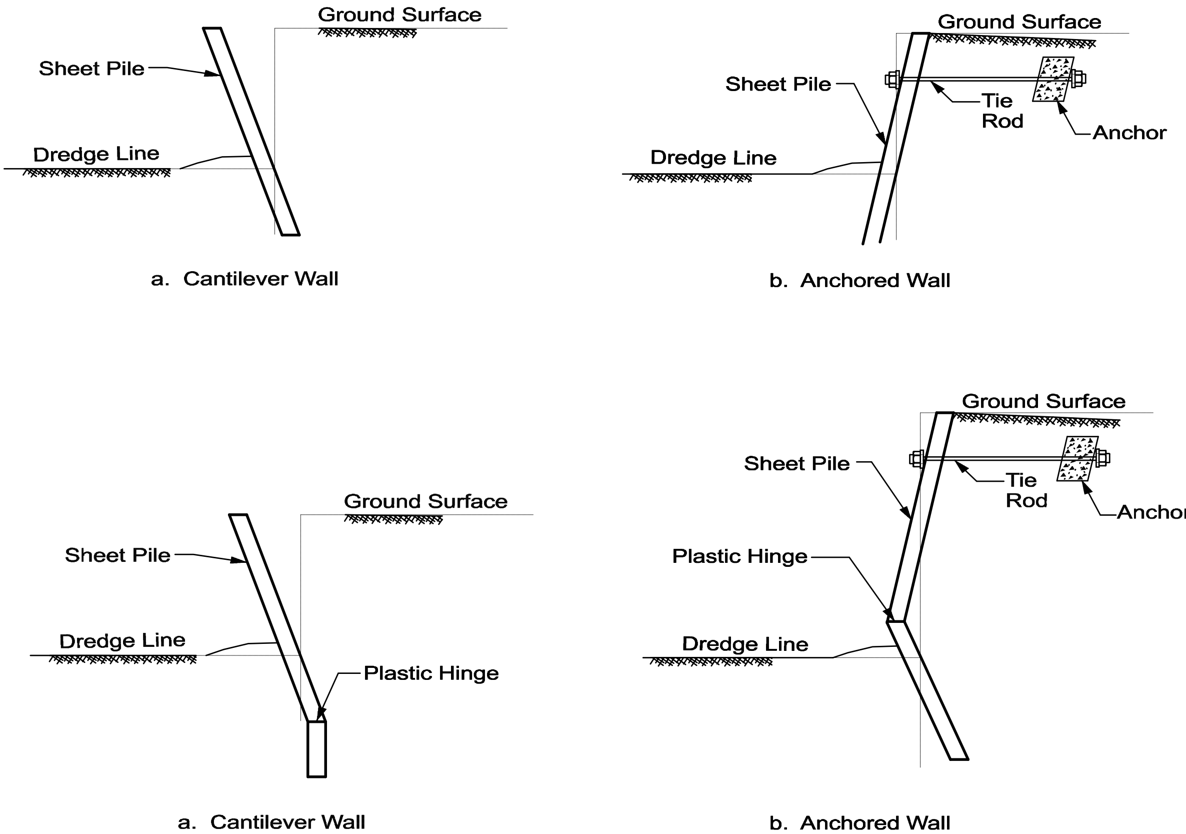


Figure 14.4-13
Flexural Failure of Non-Gravity Walls

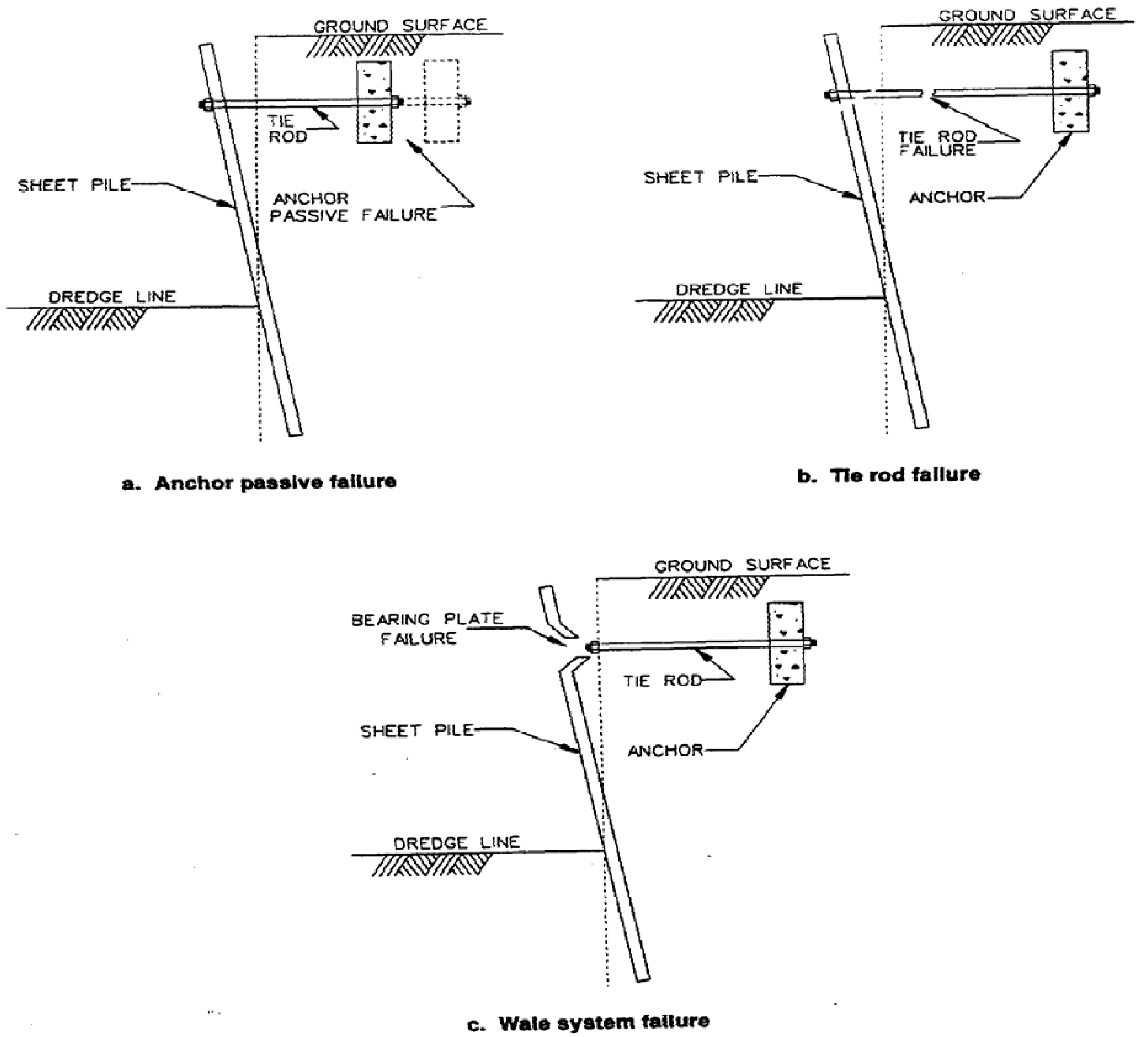


Figure 14.4-14
Flexural Failure of Non-Gravity Walls

14.4.7.2 Wall Settlement

Retaining walls shall be designed for the effects of total and differential foundation settlement at the Service I limit state, in accordance with **LRFD [11.5.2]** and 11.2. Maximum tolerable retaining wall total and differential foundation settlements are controlled largely by the potential for cosmetic and/or structural damage to facing elements, copings, barrier, guardrail, signs, pavements, utilities, structure foundations, and other highway appurtenances supported on or near the retaining wall.

14.4.7.2.1 Settlement Guidelines

The following table provides guidance for maximum tolerable vertical and total differential Settlement for various retaining wall types where Δh is the total settlement in inches and

Wall Type	Total Settlement Δh in inches	Total Differential Settlement Δh1:L (in/in)
CIP semi-gravity cantilever walls	1-2	1:500
MSE walls with large pre-cast panel facing (panel front face area >30ft^2)	1-2	1:500
MSE walls with small pre-cast panel facing (panel front face area <30ft^2)	1-2	1:300
MSE walls with full-height cast-in-panel facing	1-2	1:500
MSE walls with dry cast concrete block facing	2-4	1:200
MSE walls with geo-textile /welded-wire facing	4-8	1:50-1:60
Concrete block gravity retaining walls (wet or dry cast)	1-2	1:300
Concrete Crib walls	1-2	1:500
Bin walls	2-4	1:200
Gabion walls	4-6	1:50
Non-gravity cantilever and anchored walls	1-2.5	----

Table 14.4-3
Maximum Tolerable Settlement Guidelines for Retaining Walls

∆h1:L is the ratio of the difference in total vertical settlement between two points along the wall base to the horizontal distance between the two points(L). It should be noted that the tolerance provided in Table 14.4-3 are for guidance purposes only. More stringent tolerances may be required to meet project-specific requirements.

14.4.7.3 Overall Stability

Overall stability of the walls shall be checked at the Service I limit state using appropriate load combinations and resistance factors in accordance with **LRFD [11.6.2.3].** The stability is evaluated using limit state equilibrium methods. The Modified Bishop, Janbu or Spencer method may be used for the analysis. The analyses shall investigate all potential internal, compound and overall shear failure surfaces that penetrate the wall, wall face, bench, back-cut, backfill, and/or foundation zone. The overall stability check is performed by the Geotechnical Engineering Unit for WISDOT designed walls.

14.4.7.4 Internal Stability

Internal stability checks including anchor pullout or soil reinforcement failure and/or structural failure checks are also required as applicable for different wall systems. As an example, see Figure 14.4-11 for internal stability failure of MSE walls. Internal stability checks must be performed at Strength Limits in accordance with **LRFD [11.5.3].**

14.4.7.5 Wall Embedment

The minimum wall footing embedment shall be 1.5 ft below the lowest adjacent grade in front of the wall.

The embedment depth of most wall footings should be established below the depths the foundation soil/rock could be weakened due to the effect of freeze thaw, shrink-swell, scour, scour, erosion, erosion, construction excavation. The potential scour elevation shall be established in accordance with 11.2.2.1.1 of the Bridge Manual.

The final footing embedment depth shall be based on the required geotechnical bearing resistance, wall settlement limitations, and all internal, external, and overall (global) wall stability requirements in *AASHTO LRFD* and the *Bridge Manual*.

14.4.7.6 Wall Subsurface Drainage

Retaining wall drainage is necessary to prevent hydrostatic pressure and frost pressure. Inadequate wall sub-drainage can cause premature deterioration, reduced stability and collapse or failure of a retaining wall.

A properly designed wall sub-drainage system is required to control potentially damaging hydrostatic pressures and seepage forces behind and around a wall. A redundancy in the sub-drainage system is required where subsurface drainage is critical for maintaining retaining wall stability. This is accomplished using a pervious granular fill behind the wall.

Pipe underdrain must be provided to drain this fill. Therefore, “Pipe Underdrain Wrapped 6-Inch” is required behind all gravity retaining walls where seepage should be relieved. Gabion walls do not require a pipe drain system as these are porous due to rock fill. It is best to place the pipe underdrain at the top of the wall footing elevation. However, if it is not possible to discharge the water to a lower elevation, the pipe underdrain could be placed higher.

Pipe underdrains and weep holes may discharge water during freezing temperatures. In urban areas, this may create a problem due to the accumulation of flow and ice on sidewalks. Consideration should be given to connect the pipe underdrain to the storm sewer system.

14.4.7.7 Scour

The probable depth of scour shall be determined by subsurface exploration and hydraulic studies if the wall is located in flood prone areas. Refer to 11.2.2.1.1 for guidance related to scour vulnerability and design of walls. All walls with shallow foundations shall be located below the scour elevation.

14.4.7.8 Corrosion

All metallic components of WISDOT retaining wall systems subjected to corrosion, should be designed to last through the designed life of the walls. Corrosion protection should be designed in accordance with the criteria given in **LRFD [11.10.6].** In addition, **LRFD [11.8.7] thru [11.10]** also include design guidance for corrosion protection on non-gravity cantilever walls, anchored walls and MSE walls respectively.

14.4.7.9 Utilities

Walls that have or may have future utilities in the backfill should minimize the use of soil reinforcement. MSE, soil nail, and anchored walls commonly have conflicts with utilities and should not be used when utilities must remain in or below the reinforced soil zone unless there is no other wall option. Utilities that are encapsulated by wall reinforcement may not be accessible for replacement or maintenance. Utility agreements should specifically address future access if wall reinforcing will affect access.

14.4.7.10 Guardrail and Barrier

Guardrail and barrier shall meet the requirements of the Chapter 30 - Railings, *Facilities Development Manual,* Standard Plans, and *AASHTO LRFD*. In no case shall guardrail be placed through MSE wall or reinforced slope soil reinforcement closer than 3 ft from the back of the wall facing elements. Furthermore, the guard rail posts shall be installed through the soil reinforcement in a manner that prevents ripping, damage and distortion of the soil reinforcement. In addition, the soil reinforcement shall be designed to account for the reduced cross-section resulting from the guardrail post holes.

14.5 Cast-In-Place Concrete Cantilever Walls

14.5.1 General

A cast-in-place, reinforced concrete cantilever wall is a semi-gravity wall that consists of a base slab or footing from which a vertical wall or stem extends upward. Reinforcement is provided in both members to supply resistance to bending. These walls are generally founded on good bearing material. Cantilever walls shall not be used without pile support if the foundation stratum is prone to excessive vertical or differential settlement, unless subgrade improvements are made. Cantilever walls are typically designed to a height of 28 feet. For heights exceeding 28 feet, consideration should be given to providing a counterfort. Design of counterfort CIP walls is not covered in this chapter.

CIP cantilever walls shall be designed in accordance with *AASHTO LRFD,* design concepts presented in 14.4 and the *WisDOT Standard Specifications* including the special provisions.

14.5.2 Design Procedure for Cast-in-Place Concrete Cantilever Walls

The CIP wall shall be designed to resist lateral pressure caused by supported earth, surcharge loads and water in accordance with **LRFD [11.6].** The external stability, settlement, and overall stability shall be evaluated at the appropriate load limit states in accordance with **LRFD [11.5.5],** to resist anticipated failure mechanism. The structural components mainly stem and footing should be designed to resist flexural resistance in accordance with **LRFD [11.6.3].**

Figure 14.5-1 shows possible external stability failure and deep seated rotational failure mechanisms of CIP cantilever walls that must be investigated as part of the stability check.

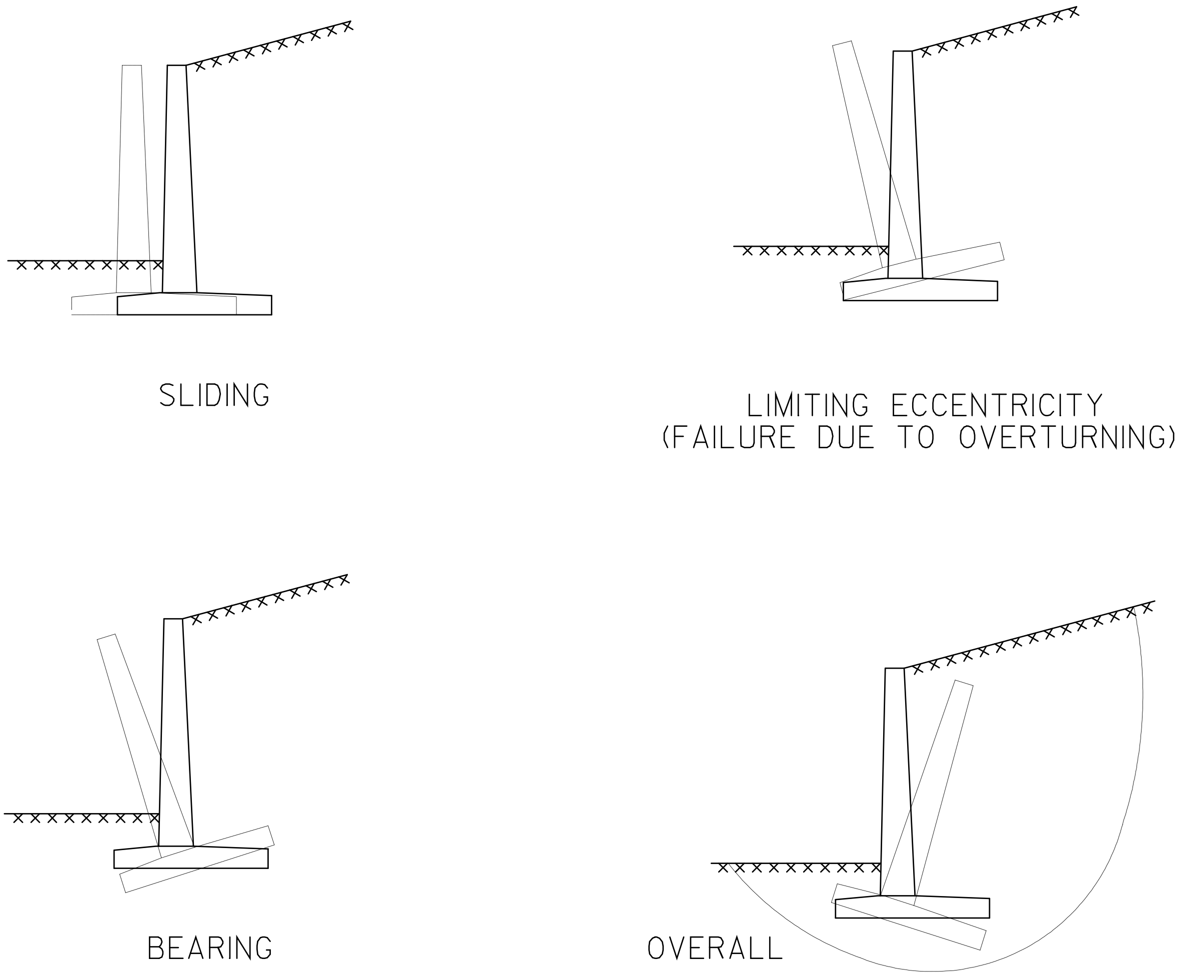


Figure 14.5-1
CIP Semi-Gravity Wall Failure Mechanism

14.5.2.1 Design Steps

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

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

2. Perform Geotechnical investigation and testing

3. Develop soil strength parameters

4. Determine preliminary sizing for external stability evaluation

5. Determine applicable unfactored or nominal loads

6. Evaluate factored loads for all appropriate limit states

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

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

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

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

14.5.3 Preliminary Sizing

A preliminary design can be performed using the following guideline.

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

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

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

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

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

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

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

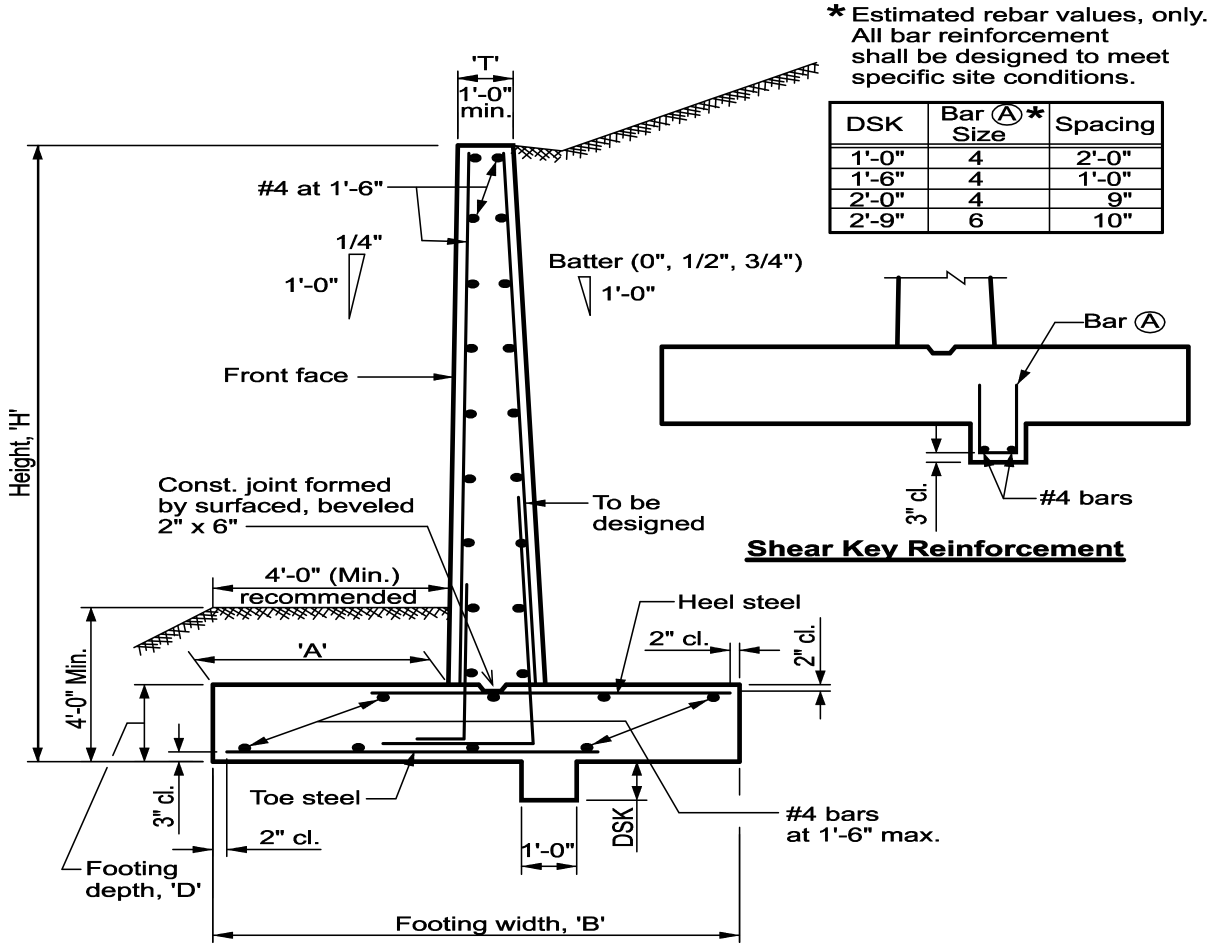


Figure 14.5-2
CIP Walls General Details

14.5.3.1 Wall Back and Front Slopes

CIP walls shall not be designed for backfill slope steeper than 2:1(H:V). Where practical, walls shall have a horizontal bench of 4.0 feet wide at the front face.

14.5.4 Unfactored and Factored Loads

Unfactored loads and moments are computed after establishing the initial wall geometry and using procedures defined in 14.4.5.4.5. A load diagram as shown in Figure14.4-1 for the earth pressure is developed assuming a triangular distribution plus additional pressures resulting from earth surcharge, water pressure, compaction or any other loads, etc. The material properties for backfill soil, concrete and steel are given in 14.4.6. The foundation and retained earth properties as recommended in the Geotechnical Report shall be used for computing nominal loads.

The computed nominal loads discussed in 14.5.4 are multiplied by applicable load factors given in Table 14.4-1. Figure 14.4-8 shows load factor and load combinations along with their application for the load limit state evaluation. A summary of load factors and load combinations as applicable for a typical CIP cantilever wall is presented in Table 14.4-1 and **LRFD [3.4.1]**, respectively. Computed factored loads and moments are used for performing stability checks.

14.5.5 External Stability Checks

The external stability check includes checks for limiting eccentricity (overturning), bearing stress, and sliding at Strength I and Extreme Event II due to vehicle impact in cases where live load traffic is carried.

14.5.5.1 Eccentricity Check

The retaining wall shall be evaluated for the eccentricity. The location of the resultant force should be within the middle half of the base width (e<B/4) of the foundation centroid for foundations on soil, and within the middle three-fourths of the base width (e< 3B/8) of the foundation centroid for foundations on rock. If there is inadequate resistance to overturning (eccentricity value greater than limits given above), consideration should be given to either increasing the width of the wall base, or providing a deep foundation.

14.5.5.2 Bearing Resistance

The Bearing resistance shall be evaluated at the strength limit state using factored loads and resistances. Bearing resistance of the walls founded directly on soil or rock shall be computed in accordance with 11.2 and **LRFD [10.6]**. The Bearing Resistance for walls on piles shall be computed in accordance with 11.3 and **LRFD [10.6]**. Figure 14.5-3 shows bearing stress criteria for a typical CIP wall on soil and rock respectively.

The vertical stress for footings on soil shall be calculated using:

$$\sigma_v = \frac{\sum V}{(B - 2e)}$$

For walls founded on rock, the vertical stress is calculated assuming a linearly distributed pressure over an effective base area. The vertical stress for footings on rock shall be computed using:

$$\sigma_v = \frac{\sum V}{B\left(1 \pm \frac{6e}{B}\right)}$$

Where

ΣV = Summation of vertical forces

B = Base width

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

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

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

$\sigma_{vmin} = 0$

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

$q_r = \phi_b q_n > \sigma_v$

Where:

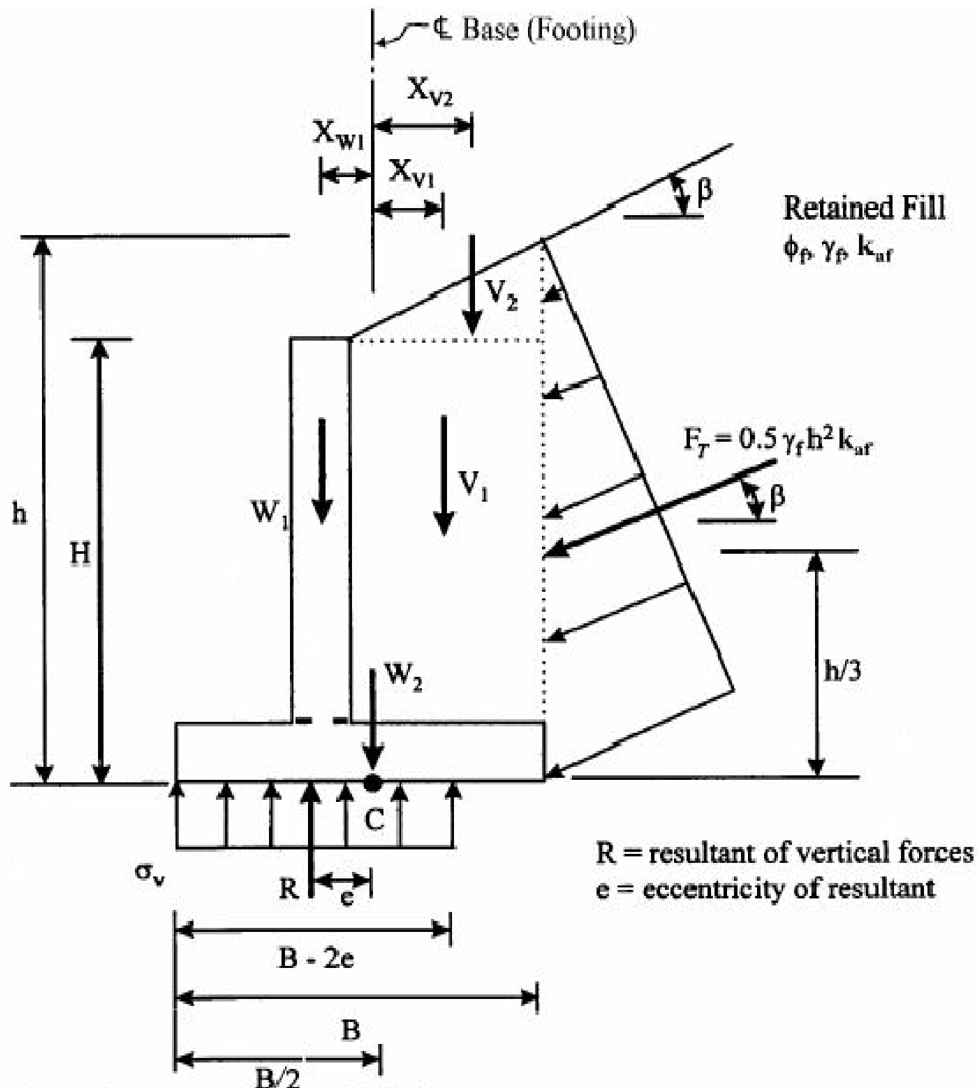
qr = Factored bearing resistance

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

σ_v = Vertical stress

B = Base width

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



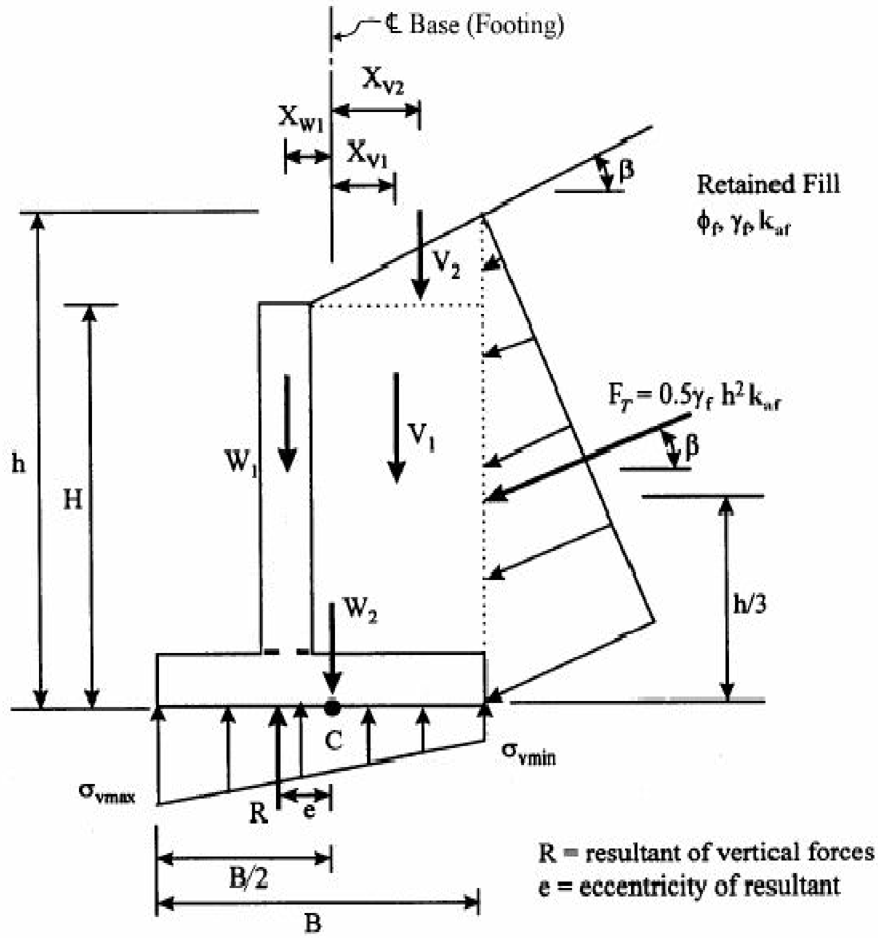
Summing moments about Point C:

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

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

Figure 14.5-3

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



If $e > B/6$, σ_{vmin} will drop to zero, and as "e" increases, the portion of the heel of the footing which has zero vertical stress increases.

Summing moments about Point C:

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

Figure 11.6.3.2-2 Bearing Stress Criteria for Conventional Wall Foundations on Rock.

Figure 14.5-4
Loading Diagram and Bearing Stress Criteria for CIP Cantilever Walls on Rock
(source *AASHTO LRFD*)

14.5.5.3 Sliding

The sliding resistance of CIP cantilever walls is computed by considering the wall as a shallow footing resting on soil/rock or footing resting on piles in accordance with **LRFD [10.5]**. Sliding resistance of a footing resting on soil/rock foundation is computed in accordance with the **LRFD [10.6.3.4]** using the equation given below:

$$R_R = \phi\, R_n = \phi_\tau\, R_\tau + \phi_{ep} R_{ep}$$

Where:

R_R = Factored resistance against failure by sliding

R_n = Nominal sliding resistance against failure by sliding

ϕ_τ = Resistance factor for shear between soil and foundation per **LRFD [Table 10.5.5.2.2.1]**

R_τ = Nominal sliding resistance between soil and foundation

ϕ_{ep} = Resistance factor for passive resistance per **LRFD Table [10.5.5.2.2.1]**

R_{ep} = Nominal passive resistance of soil throughout the life of the structure

Contribution from passive earth pressure resistance against the embedded portion of the wall is neglected if the soil in front of the wall can be removed or weakened by scouring, erosion or any other means. Also, the live load surcharge is not considered as a stabilizing force over the heel of the wall when checking sliding.

If adequate sliding resistance cannot be achieved, footing design may be modified as follows:

- Increase the base width of the footing
- Construct a shear key
- Increase wall embedment to a sufficient depth, where passive resistance can be relied upon
- Incorporate a deep foundation, including battered piles (Usually a costly measure)

Guideline for selecting the shear key design is presented in 14.5.7.3. The design of wall footings resting on piles is performed in accordance with **LRFD [10.5]** and Chapter 11 - Foundation Support. Footings on piles resist sliding by the following:

1. Passive earth pressure in front of wall. Same as spread footing.
2. Lateral resistance of vertical piles as well as the horizontal components of battered piles. Maximum batter is 3 inches per foot. Refer to Chapter 11 - Foundation Support for lateral load capacity of piles.

3. Lateral resistance of battered or vertical piles in addition to horizontal component of battered piles. Refer to Chapter 11- Foundation Support for allowable lateral load capacity.

4. Do not use soil friction under the footing as consolidation of the soil may eliminate contact between the soil and footing.

14.5.5.4 Settlement

The settlement of CIP cantilever walls can be computed in accordance with guidelines and performance criteria presented in 14.4.7.2. The guideline for total and differential settlement is presented in Table 14.4-3. The actual performance limit can be changed for specific project requirements. For additional guidance contact the Geotechnical Engineering Unit.

14.5.6 Overall Stability

Investigate Service 1 load combination using an appropriate resistance factor and procedures discussed in **LRFD [11.6]** and 14.4.7.3. In general, the resistance factor, φ, may be taken as;

- 0.75 - where the geotechnical parameters are well defined, and slope does not support or contain a structural element.

- 0.65 – where the geotechnical parameters are based on limited information or the slope contains or supports a structural element.

14.5.7 Structural Resistance

The structural design of the stem and footing shall be performed in accordance with *AASHTO LRFD* and the design guidelines discussed below.

14.5.7.1 Stem Design

The initial sizing of the stem should be selected in accordance with criteria presented in 14.5.3. The stems of cantilever walls shall be designed as cantilevers supported at the footing. Axial loads (including the weight of the wall stem and frictional forces due to backfill acting on the wall stem) shall be considered in addition to the bending due to eccentric vertical loads, surcharge loads and lateral earth pressure if they control the design of the wall stems. The flexural design of the cantilever wall should be performed in accordance with *AASHTO LRFD*.

Loads from railings or parapets on top of the wall need not be applied simultaneously with live loads. These are dynamic loads which are resisted by the mass of the wall.

14.5.7.2 Footing Design

The footing of a cantilever wall shall be designed as a cantilever beam. The heel section must support the weight of the backfill soil and the shear component of the lateral earth pressure. All loads and moments must be factored using the criteria load factors discussed in 14.5.4. Use the following criteria when designing the footing.

1. Minimum footing thickness shall be selected in accordance with criteria presented in 14.5.3. The final footing thickness shall be based on shear at a vertical plane behind the stem.

2. For toe, design for shear at a distance from the face of the stem equal to the effective "d" distance of the footing. For heel, design for shear at the face of stem.

3. Where the footing is resting on piles, the piles shall be designed in accordance with criteria for pile design presented in Chapter 11 – Foundation Support. Embed piles six inches into footing. Place bar steel on top of the piles.

4. For spread footings, use a minimum of 3 inches clear cover at the bottom of footing. Use 2 inches clear cover for edge distance.

5. The critical sections for bending moments in footings shall be taken at the front and back faces of the wall stem. Bearing pressure along the bottom of the heel extension may conservatively be ignored. No bar steel is provided if the required area per foot is less than 0.05 square inches.

6. Design for heel moment, without considering the upward soil or pile reaction, is not required unless such a condition actually exists.

14.5.7.3 Shear Key Design

A shear key shall be provided to increase the sliding resistance when the factored sliding resistance determined using procedure discussed in 14.5.5.3 is inadequate. Use the following criteria when designing the shear key:

1. Place shear key in line with stem except under severe loading conditions.

2. The key width is 1'-0" in most cases. The minimum key depth is 1'-0".

3. Place shear key in unformed excavation against undisturbed material.

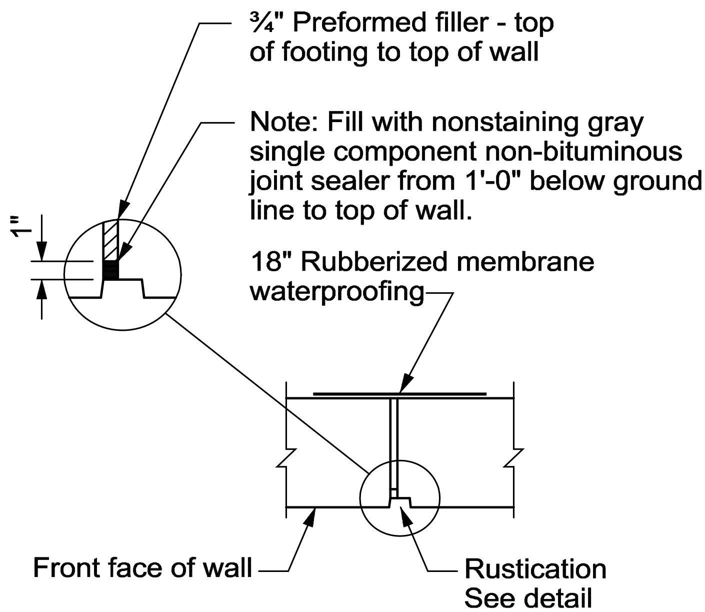
4. Analyze shear key in accordance with **LRFD [10.6.3.4]** and 14.5.5.3 .

5. The shape of shear key in rock is governed by the quality of the rock, but in general a 1 ft. by 1 ft key is appropriate.

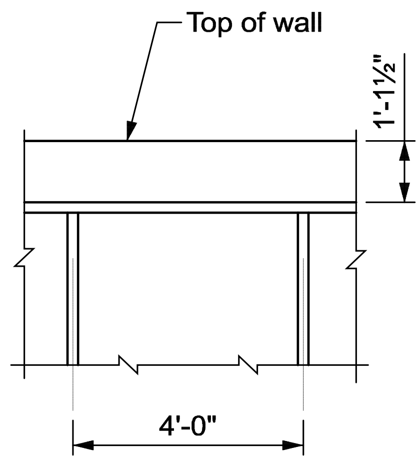
14.5.7.4 Miscellaneous Design Information

1. Contraction joints shall be provided at intervals not exceeding 30 feet and expansion joints at intervals not exceeding 90 feet for reinforced concrete walls. Typical details of expansion and contraction joints are given in Figure 14.5-5. Expansion joints shall be constructed with a joint, filling material of the appropriate thickness to ensure the

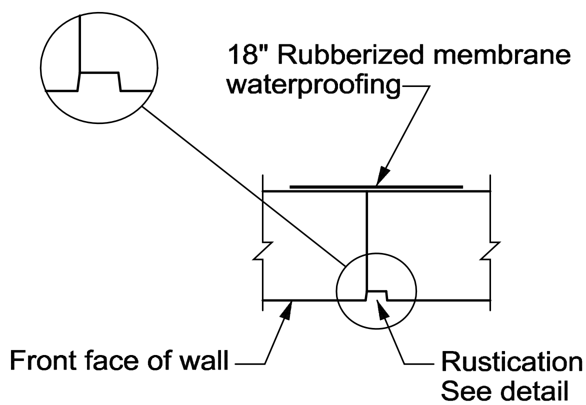
functioning of the joint and shall be provided with a waterstop capable of functioning over the anticipated range of joint movements.



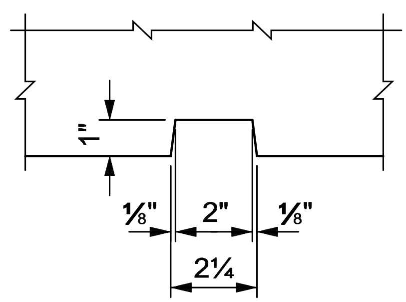
Vertical Expansion Joint
Do not run any bar steel thru joint
Max. spacing of joint = 90'



Elevation Of Wall



Vertical Contraction Joint
Do not run any bar steel thru joint
Max. spacing of joint = 30'



Rustication Detail
Typical horizontal and vertical

Figure 14.5-5
Retaining Wall Joint Details

2. Optional transverse construction joints are permitted in the footing, with a minimum spacing of three panel lengths. Footing joints should be offset a minimum of 1'-0 from wall joints. Run reinforcing bar steel thru footing joints.

3. The backfill material behind all cantilever walls shall be granular, free draining, non expansive, non-corrosive material and shall be drained by weep holes with pervious material or other positive drainage systems, placed at suitable intervals and elevations. Granular backfill is placed behind the wall only to a vertical plane 18 inches beyond the face of footing. Lower limit is to the bottom of the footing.

4. If a wall is adjacent to a traveled roadway or sidewalk, use pipe underdrains in back of the wall instead of weep holes. Use a six-inch pipe wrapped underdrain located as detailed in this chapter. Provide a minimum slope of 0.5% and discharge to suitable drainage (i.e. a storm sewer system or ditch).

14.5.8 Design Tables for Cast-in-Place Concrete Cantilever Walls

Design tables suitable for use in preliminary design have been assembled and presented in this sub-section. These design tables are based on WisDOT design criteria and the material properties summarized in Table 14.5-1. Active earth pressure for the design tables was computed using the Rankine's equation for horizontal slopes and Coulomb's equation for surcharged slopes with the resultant perpendicular to the wall backface plus the wall friction angle. It was assumed that no water pressure exists. Service limit states were ignored in the analyses. The requirement of concrete is in accordance with **LRFD [5.4.2]** and 9.2. The requirement for bar steel is based on **LRFD [5.4.3]** and 9.3. The aforementioned assumptions were used in creating Table 14.5-2 thru Table 14.5-7. Refer to Figure 14.5-2 for details.

These tables should not be used if any of the assumptions or strength properties of the retained or foundation earth or the materials used for construction are different than those used in these design tables. The designer should also determine if the long-term or short-term soil strength parameters govern external stability analyses.

14.5.9 Design Examples

Refer to 14.18 for the design examples.

Design Criteria/Assumptions	Value
Concrete strength	3.5 ksi
Reinforcement yield strength	60 ksi
Concrete unit weight	150 pcf
Soil unit weight	120 pcf
Friction angle between fill and wall	21 degrees
Angle of Internal Friction (Soil - Backfill)	30 degrees
Angle of Internal Friction (Soil - Foundation)	34 degrees
Angle of Internal friction (Rock)	25 degrees
Cohesion (Soil)	0 psi
Cohesion (Rock)	20 psi
Soil Cover over Footing	4 feet
Stem Front Batter	0.25"/ft
Stem Back Batter	See Tables
Factored bearing resistance (On Soil)	**LRFD [10.6.3.1.2]**
Factored bearing resistance (On Rock)	20 ksf
Live Load Surcharge (Traffic)	240 psf
Live Load Surcharge (No Traffic)	100 psf
Lateral Earth Pressure (Horizontal Backfill)	Rankine
Lateral Earth Pressure (2:1 Backfill)	Coulomb

Table 14.5-1
Assumptions Summary for Preliminary Design of CIP Walls

HORIZONTAL BACKFILL – NO TRAFFIC – ON SOIL

H (ft)	B (ft)	A (ft)	D (ft)	Batter (in/ft)	Toe Steel Size	Toe Steel Spa	Toe Steel L	Heel Steel Size	Heel Steel Spa	Heel Steel L	Stem Steel Size	Stem Steel Spa	Shear Key	DSK
6	3'- 6"	0'- 9"	1'- 6"	0	---	---	---	---	---	---	---	---	NO	---
8	4'- 6"	1'- 0"	1'- 6"	0	---	---	---	4	12	3' - 5"	4	12	NO	---
10	5'- 3"	1'- 3"	1'- 6"	0	---	---	---	4	12	3' - 10"	4	12	NO	---
12	6'- 3"	1'- 6"	2'- 0"	0	---	---	---	4	10	4' - 7"	5	12	NO	---
14	7'- 3"	1'- 9"	2'- 0"	0	4	12	2' - 7"	5	9	5' - 6"	6	10	NO	---
16	8'- 0"	2'- 0"	2'- 0"	0.50	4	12	2' - 10"	5	8	5' - 5"	6	10	NO	---
18	8'- 9"	2'- 3"	2'- 0"	0.50	4	12	3' - 1"	7	11	6' - 7"	6	8	NO	---
20	9'- 9"	2'- 6"	2'- 0"	0.50	4	10	3' - 4"	7	8	7' - 3"	7	8	NO	---
22	10'- 6"	2'- 9"	2'- 3"	0.50	4	9	3' - 7"	9	12	9' - 2"	9	12	NO	---
24	11'- 6"	3'- 0"	2'- 9"	0.50	4	9	3' - 10"	9	11	9' - 10"	8	9	NO	---
26	12'- 0"	4'- 0"	2'- 9"	0.50	5	8	4' - 10"	8	8	8' - 5"	8	8	YES	1'- 6"
28	13'- 0"	5'- 0"	3'- 0"	0.75	7	11	6' - 6"	8	8	7' - 9"	8	7	YES	1'- 6"

Table 14.5-2

Reinforcement for Cantilever Retaining Walls

HORIZONTAL BACKFILL – TRAFFIC – ON SOIL

H (ft)	B (ft)	A (ft)	D (ft)	Batter (in/ft)	Toe Steel Size	Toe Steel Spa	Toe Steel L	Heel Steel Size	Heel Steel Spa	Heel Steel L	Stem Steel Size	Stem Steel Spa	Shear Key	DSK
6	4'- 6"	0'- 6"	1'- 6"	0	---	---	---	4	12	3' - 11"	---	---	NO	---
8	5'- 3"	0'- 9"	1'- 6"	0	---	---	---	4	11	4' - 5"	4	12	NO	---
10	6'- 6"	1'- 0"	1'- 6"	0	---	---	---	6	12	5' - 11"	4	8	NO	---
12	7'- 3"	1'- 3"	2'- 0"	0	---	---	---	6	11	6' - 5"	5	9	NO	---
14	8'- 3"	1'- 6"	2'- 0"	0	---	---	---	7	10	7' - 7"	6	9	NO	---
16	9'- 0"	2'- 3"	2'- 0"	0.50	4	12	3'- 1"	7	10	7'- 0 "	6	9	NO	---
18	9'- 3"	2'- 9"	2'- 0"	0.50	4	10	3'- 7"	7	10	6' - 7"	8	12	YES	1'- 0"
20	10'- 0"	3'- 6"	2'- 0"	0.50	5	9	4'- 4"	6	7	6'- 0 "	8	10	YES	1'- 0"
22	11'- 0"	4'- 3"	2'- 3"	0.50	5	7	5'- 1"	6	7	6' - 2"	7	7	YES	1'- 0"
24	11'- 9"	5'- 0"	2'- 6"	0.50	7	10	6'- 6"	6	7	6'- 0 "	9	11	YES	1'- 6"
26	12'- 9"	5'- 9"	2'- 9"	0.50	8	11	7'- 9"	6	7	6' - 2"	9	9	YES	1'- 6"
28	14'- 3"	7'- 0"	3'- 0"	0.75	9	11	9'- 7"	6	7	5' - 9"	9	9	YES	2'- 0"

Table 14.5-3

Reinforcement for Cantilever Retaining Walls

2:1 BACKFILL – NO TRAFFIC – ON SOIL

H	B	A	D	Batter	Toe Steel			Heel Steel			Stem Steel		Shear	
(ft)	(ft)	(ft)	(ft)	(in/ft)	Size	Spa	L	Size	Spa	L	Size	Spa	Key	DSK
6	4'- 6"	2'- 0"	1'- 6"	0	---	---	---	---	---	---	4	12	YES	1'- 0"
8	6'- 0"	2'- 6"	1'- 6"	0	4	12	3'- 4"	4	12	3' - 5"	4	9	YES	1'- 0"
10	7'- 6"	2'- 0"	1'- 6"	0	4	12	2'- 10"	6	11	5' - 11"	6	9	YES	1'- 0"
12	9'- 0"	1'- 9"	2'- 0"	0	4	12	2'- 7"	7	9	8' - 2"	8	11	YES	1'- 0"
14	10'- 6"	2'- 6"	2'- 6"	0	4	12	3'- 4"	8	10	9' - 8"	9	10	YES	1'- 6"
16	12'- 3"	3'- 9"	2'- 9"	0.50	5	12	4'- 7"	7	7	8' - 10"	9	10	YES	2'- 0"
18	14'- 0"	4'- 6"	3'- 0"	0.50	6	12	5'- 7"	9	9	11' - 2"	10	10	YES	2'- 0"
20	15'- 6"	5'- 6"	3'- 3"	0.50	7	11	7'- 0"	10	11	12' - 8"	10	8	YES	2'- 9"

Table 14.5-4
Reinforcement for Cantilever Retaining Walls

HORIZONTAL BACKFILL – NO TRAFFIC – ON ROCK

H	B	A	D	Batter	Toe Steel			Heel Steel			Stem Steel	
(ft)	(ft)	(ft)	(ft)	(in/ft)	Size	Spa	L	Size	Spa	L	Size	Spa
6	2'- 9"	0'- 9"	1'- 6"	0	---	---	---	---	---	---	4	12
8	3'- 6"	1'- 0"	1'- 6"	0	---	---	---	---	---	---	4	12
10	4'- 3"	1'- 3"	1'- 6"	0	---	---	---	4	12	2' - 10"	4	12
12	5'- 0"	1'- 6"	2'- 0"	0	4	12	2'- 4"	4	12	3' - 4"	5	12
14	5'- 9"	1'- 9"	2'- 0"	0	4	12	2'- 7"	4	12	3' - 10"	6	10
16	6'- 6"	2'- 0"	2'- 0"	0.50	4	12	2'- 10"	4	11	3' - 8"	6	10
18	7'- 3"	2'- 3"	2'- 0"	0.50	4	11	3'- 1"	5	12	4' - 3"	6	8
20	7'- 9"	2'- 6"	2'- 0"	0.50	5	11	3'- 4"	5	9	4' - 5"	8	11
22	8'- 6"	2'- 9"	2'- 0"	0.50	5	9	3'- 7"	6	10	5' - 1"	7	7
24	9'- 3"	3'- 0"	2'- 0"	0.50	6	10	4'- 1"	7	10	6'- 0 "	9	11
26	10'- 0"	3'- 3"	2'- 3"	0.50	6	9	4'- 4"	8	11	7' - 2"	10	12
28	10'- 6"	3'- 6"	2'- 6"	0.75	6	8	4'- 7"	8	11	6' - 9"	9	9

Table 14.5-5
Reinforcement for Cantilever Retaining Walls

HORIZONTAL BACKFILL – TRAFFIC – ON ROCK

H	B	A	D	Batter	Toe Steel			Heel Steel			Stem Steel	
(ft)	(ft)	(ft)	(ft)	(in/ft)	Size	Spa	L	Size	Spa	L	Size	Spa
6	3'- 6"	0'- 9"	1'- 6"	0	---	---	---	---	---	---	4	12
8	4'- 3"	1'- 0"	1'- 6"	0	---	---	---	4	12	3' - 2"	4	12
10	5'- 0"	1'- 3"	1'- 6"	0	---	---	---	4	12	3' - 7"	4	8
12	5'- 9"	1'- 6"	2'- 0"	0	---	---	---	4	12	4' - 1"	5	9
14	6'- 6"	1'- 9"	2'- 0"	0	4	12	2'- 7"	4	8	4' - 6"	6	9
16	7'- 3"	2'- 0"	2'- 0"	0.50	4	12	2'- 10"	4	7	4' - 5"	7	12
18	8'- 0"	2'- 3"	2'- 0"	0.50	4	11	3'- 1"	6	11	5' - 4"	8	12
20	8'- 9"	2'- 6"	2'- 3"	0.50	4	9	3'- 4"	6	9	5' - 9"	8	10
22	9'- 6"	2'- 9"	2'- 6"	0.50	5	12	3'- 7"	7	11	6' - 8"	9	12
24	10'- 3"	3'- 0"	2'- 9"	0.50	5	10	3'- 10"	7	9	7' - 1"	9	11
26	11'- 0"	4'- 0"	2'- 6"	0.50	7	10	5'- 6"	8	11	7' - 5"	8	7
28	11'- 9"	4'- 3"	2'- 9"	0.75	6	7	5'- 4"	8	11	7' - 3"	8	7

Table 14.5-6
Reinforcement for Cantilever Retaining Walls

2:1 BACKFILL – NO TRAFFIC – ON ROCK

H	B	A	D	Batter	Toe Steel			Heel Steel			Stem Steel	
(ft)	(ft)	(ft)	(ft)	(in/ft)	Size	Spa	L	Size	Spa	L	Size	Spa
6	3'- 9"	2'- 0"	1'- 6"	0	---	---	---	---	---	---	4	12
8	5'- 0"	2'- 9"	1'- 6"	0	4	12	3'- 7"	4	12	2' - 2"	4	12
10	6'- 0"	3'- 3"	1'- 6"	0	4	9	4'- 1"	4	12	2' - 7"	6	12
12	7'- 0"	4'- 0"	2'- 0"	0	5	11	4'- 10"	4	12	2' - 10"	6	9
14	8'- 3"	4'- 6"	2'- 0"	0	6	10	5'- 7"	4	12	3' - 7"	8	11
16	9'- 0"	5'- 3"	2'- 0"	0.50	8	11	7'- 3"	4	12	2' - 11"	8	11
18	10'- 0"	4'- 9"	2'- 0"	0.50	8	10	6'- 9"	6	11	4' - 10"	9	10
20	11'- 3"	4'- 0"	2'- 6"	0.50	7	10	5'- 6"	8	10	8'- 0 "	11	11
22	12'- 3"	4'- 6"	3'- 0"	0.50	7	9	6'- 0"	9	12	9' - 2"	11	9

Table 14.5-7
Reinforcement for Cantilever Retaining Walls

14.5.10 Summary of Design Requirements

1. Stability Check

   a. Strength I and Extreme Event II limit states

      - Eccentricity check
      - Bearing Stress check
      - Sliding

   b. Service I limit states

      - Overall Stability
      - Settlement

2. Foundation Design Parameters

   Use values provided by Geotechnical analysis

3. Concrete Design Data

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

4. Retained Soil

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

5. Soil Pressure Theory

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

6. Surcharge Load

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

7. Load Factors

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

Table 14.5-8
Load Factor Summary for CIP Walls

8. Bearing Resistance Factors

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

   - ϕ_b = 0.45 **LRFD Table [10.5.5.2.2-1]**

9. Sliding Resistance Factors

   For SPT test and Footing on soils and Rock

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

10. Sliding Resistance Factors

   For SPT test and Footing on soils and Rock

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

14.6 Mechanically Stabilized Earth Retaining Walls

14.6.1 General Considerations

Mechanically Stabilized Earth (MSE) is the term used to describe the practice of reinforcing a mass of soil with either metallic or geosynthetic soil reinforcement which allows the mass of soil to function as a gravity retaining wall structure. The soil reinforcement is placed horizontally across potential planes of shear failure and develops tension stresses to keep the soil mass intact. The soil reinforcement is attached to a wall facing located at the front face of the wall.

The design of MSE walls shall meet the *AASHTO LRFD* requirements in accordance with 14.4.2. The service life requirement for both permanent and temporary MSE wall systems is presented in 14.4.3.

The MSE walls shall be designed for external stability of the wall system and internal stability of the reinforced soil mass. The overall stability shall also be considered as part of design evaluation. MSE walls are proprietary wall systems and the design responsibilities with respect to overall, external, and internal stability is shared between the designer and contractor. The designer is responsible for the overall, and external stability, whereas the contractor is responsible for the internal stability and structural design of the wall. The responsibilities of the designer and contractor are outlined in 14.6.3.2. The design of MSE walls provided by the contractor must also be in compliance with the WisDOT special provisions as stated in 14.15.2 and 14.16

The guidelines provided herein for MSE walls do not apply to geometrically complex MSE wall systems such as tiered walls (walls stacked on top of one another), back-to-back walls, or walls which have trapezoidal sections. Design guidelines for these cases are provided in publications FHWA-NHI-10-024 and FHWA-NHI-10-025.

Horizontal alignment and grades at the bottom and top of the wall are determined by the design engineer. The design must be in compliance with the WisDOT special provisions for the project and the policy and procedures as stated in the *Bridge Manual* and *FDM*.

14.6.1.1 Usage Restrictions for MSE Walls

Construction of MSE walls with either block or panel facings should not be used when any of the following conditions exist:

1. If the available construction limit behind the wall does not meet the soil reinforcement length requirements.

2. Sites where extensive excavation is required or sites that lack granular soils and the cost of importing suitable fill material may render the system uneconomical.

3. At locations where erosion or scour may undermine or erode the reinforced fill zone or any supporting leveling pad.

4. Soil is contaminated by corrosive material such as acid mine, drainage, other industrial pollutants, or any other condition which increases corrosion rate, such as the presence of stray electrical currents.

5. There is potential for placing buried utilities within (or below) the reinforced zone unless access is provided to utilities without disrupting reinforcement and breakage or rupture of utility lines will not have a detrimental effect on the stability of the wall. Contact WisDOT's Structures Design Section.

In addition to the above, MSE Walls with block facings should not be used if any of the following condition exist:

6. The wall is a component of an abutment structure, either a wingwall or a wall parallel to C/L of bearing.

7. If a railing or fence is placed behind the wall, the posts cannot be driven thru the reinforcement as it will misalign the facing. The reinforcement must be cut and holes made or preferably a blockout is provided. This only applies to the top layer which should be no more than 24 inches below the top of the finished grade. Penetration thru the second layer of reinforcement following conditions exist:

8. The wall is a component of an abutment structure, either a wingwall or a wall parallel to the C/L of bearing.

9. Traffic barriers or roadway pavements are vertically supported by the wall.

14.6.2 Structural Components

The main structural elements or components of an MSE wall are discussed below. General elements of a typical MSE wall are shown in Figure 14.6-1. These include:

- Reinforced Earthfill
- Reinforcement
- Wall Facing Element
- Leveling Pad
- Wall Drainage

A combination of different wall facings and reinforcement provide a choice of selecting an MSE wall which can be used for several different functions.

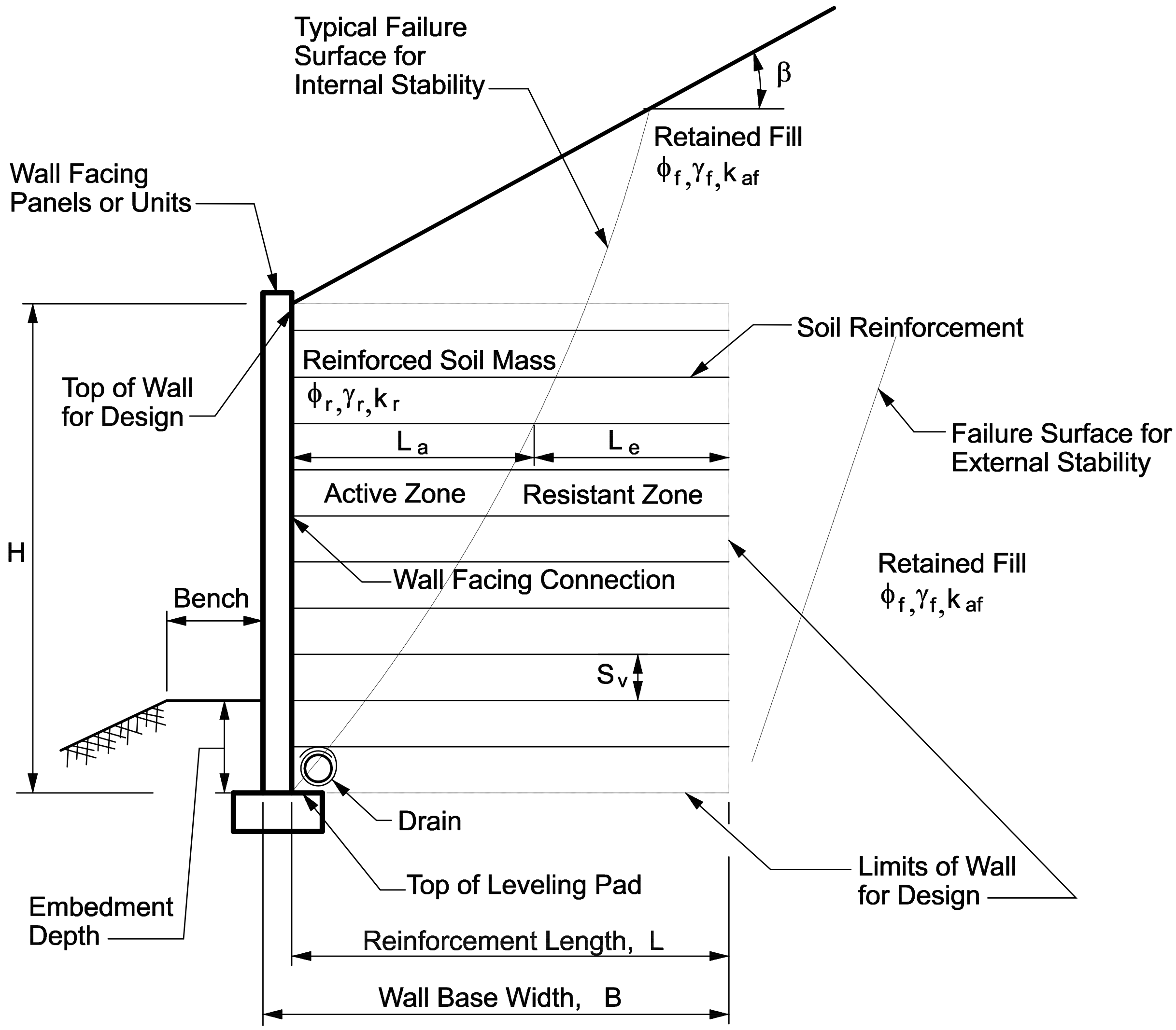


Figure 14.6-1
Structural Components of MSE Walls

14.6.2.1 Reinforced Earthfill Zone

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

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

Table 14.6-1

Electrochemical Properties of Reinforced Fill MSE Walls

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

14.6.2.2 Reinforcement:

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

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

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

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

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

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

The non-metallic or extensible reinforcement includes the following:

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

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

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

14.6.2.3 Facing Elements

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

Major facing types are:

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

Segmental Precast Concrete Panels

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

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

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

Concrete Modular Blocks Facings

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

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

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

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

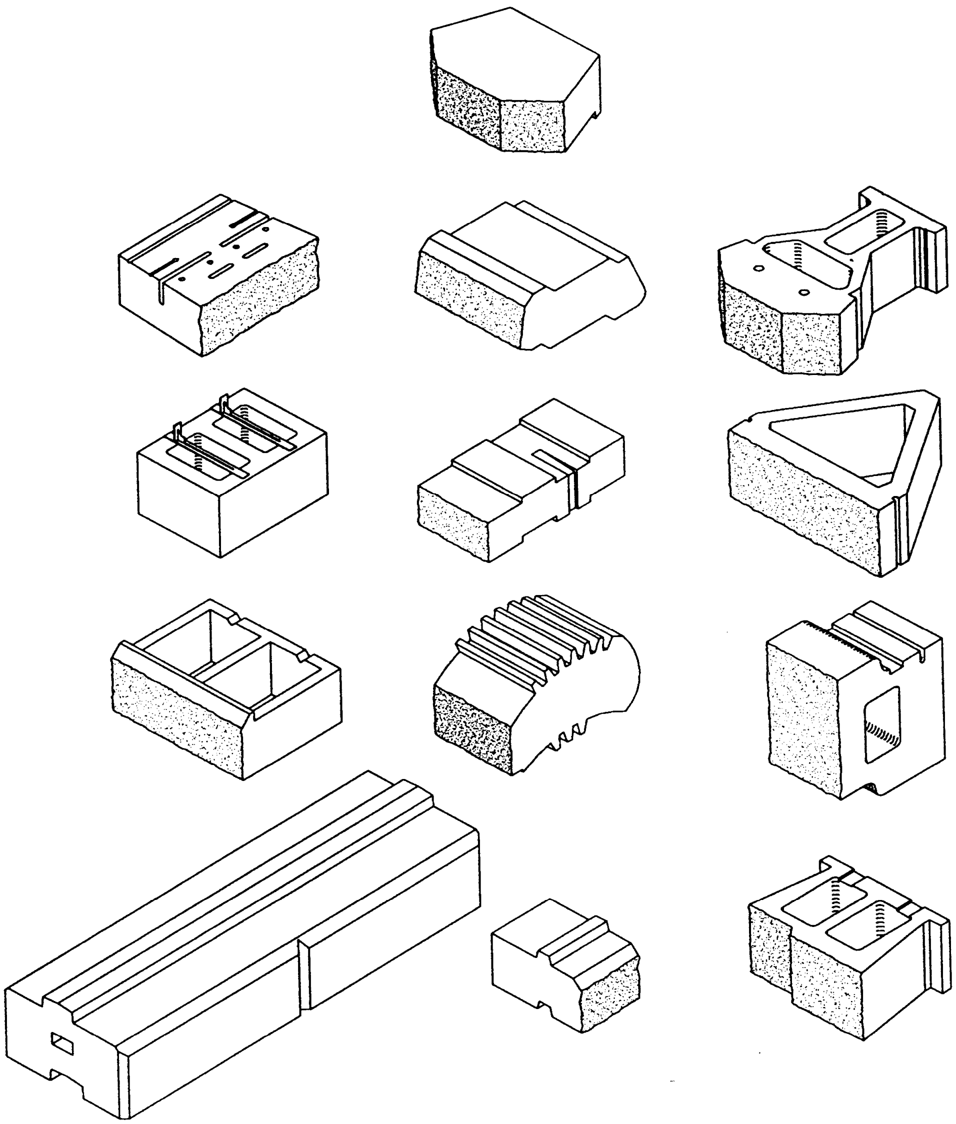
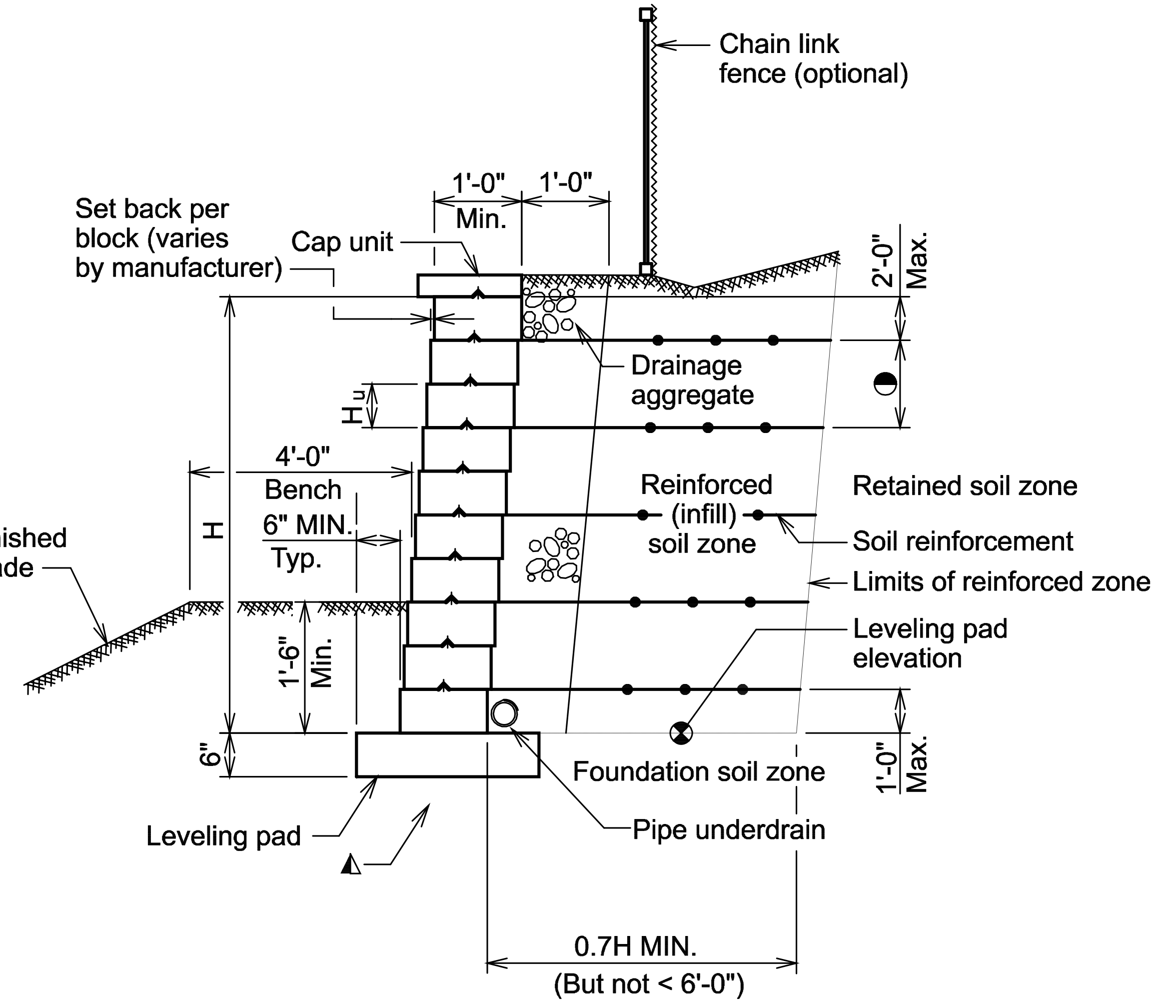


Figure 14.6-2
Modular Blocks
(Source FHWA-NHI-10-025)



Modular Block MSE Wall

Ground improvement meaures should be taken when the soil below the levelling pad is poor or subject to frost heave.

Maximum vertical spacing of soil reinforcement layers shall be two times the block depth (Hu) or 32 inches, whichever is less.

Figure 14.6-3
Typical Modular Block MSE Walls

MSE Wire-Faced Facing

Welded wire fabric facing is used to build MSE wire-faced walls. These are essentially MSE walls with a welded wire fabric facing instead of a precast concrete facing. The wire size, spacing and patterns used in the facing are developed from performance data of full size wall tests and from applications in actual walls. A test to determine the connection strength between the soil reinforcement and the facing panels is required. Some systems do not use a connection because the ground reinforcement and facing panel are of one piece construction.

MSE wire-faced wall systems usually incorporate a backing mat behind the front facing. A fine metallic screen or filter fabric is placed behind the backing mat (or behind the facing if a backing mat is not used) to prevent the backfill from passing thru the front face.

MSE wire-faced walls can tolerate considerable differential settlement because of the flexibility of the wire facing. The limiting differential settlement is 1/50. The flexibility of the wire facing results in face bulging between ground reinforcement. The actual amount varies per system but normally is less than one inch. Recommended limits on bulging are 2" for permanent walls and 3" for temporary walls.

When MSE wire-faced walls are used for permanent wall applications, all steel components must be galvanized. When used for temporary wall applications black steel (non-galvanized) may be used since the walls are usually left in place and buried.

Temporary MSE wire-faced walls can be used as temporary shoring if site conditions permit. This wall type can also be used when staged construction is required to maintain traffic when an existing roadway is being raised and/or widened in conjunction with bridge approaches, railroad crossings or road reconstruction.

Cast-In- Place Concrete Facing

MSE walls with cast in place concrete facings are identical to MSE wire faced walls except a cast-in-place concrete facing is added after the wire face wall is erected. Modifications are made to the standard wire face wall detail to anchor the concrete facing to the wire facing and soil reinforcement. They are usually used when a special aesthetic facial treatment is required without the numerous joints that are common to precast panels. They can also be used where differential settlement is above tolerable limits for other wall types. A MSE wire faced wall can be constructed and allowed to settle with the concrete facing added after consolidation of the foundation soils has occurred.

The cast-in-place concrete facing shall be a minimum of 8-inches thick and contain coated or galvanized reinforcing steel. This is required because the panels and/or anchor that extend into the cast-in-place concrete are galvanized and a corrosion cell would be created if black steel contacts galvanized steel. All wire ties and bar chairs used in the cast–in-place concrete must also be coated or galvanized. Note that the 8-inch minimum wall thickness will occur at the points of maximum panel bulging and that the wall will be thicker at other locations. Also note that the 8-inch minimum is measured from the trough of any form liner or rustication.

Vertical construction joints are required in the cast-in-place concrete facing to allow for expansion and contraction and to allow for some differential settlement. Closer spacing of vertical construction joints is required when differential settlement may occur, but by delaying the placement of the cast-in-place concrete, the effects of differential settlement is minimized. Higher walls also require closer spacing of vertical construction joints if differential settlement is anticipated. Horizontal construction joints may disrupt the flow of a special aesthetic facial treatment and are sometimes not allowed for that reason. The designer should specify if optional horizontal construction joints are allowed. Cork filler is placed at vertical construction joints because cork is compressible and will allow some expansion and rotation to occur at the joint. An expandable polyvinyl chloride waterstop (PCW) is used on the back side of a vertical construction joint. Since forms are only used at the front face of the wall the PCW can be attached to a 10-inch board which is supported by the wire facing. The 8-inch minimum wall thickness may be decreased at the location of the vertical construction joint to accommodate the PCW and its support board.

Geosynthetic Facing

Geosynthetic reinforcements are looped around at the facing to form the exposed face of the MSE Wall. These faces are susceptible to ultraviolet light degradation, vandalism, and damage due to fire. Geogrid used for soil reinforcement can be looped around to form the face of the completed retaining structure in a similar manner to welded wire mesh and fabric facing. This facing is generally used in temporary applications. Similar to wire faced walls, these walls typically have a geotextile behind the geogrids, to prevent material from passing through the face.

14.6.3 Design Procedure

14.6.3.1 General Design Requirements

The procedure for design of an MSE wall requires evaluation of external stability and internal stability (structural design) at Strength Limit States and overall stability and vertical/lateral movement at Service Limit State. The Extreme Event II load combination is used to design and analyze for vehicle impact where traffic barriers are provided to protect MSE walls. The design and stability is performed in accordance with *AASHTO LRFD* and design guidance discussed in 14.4.

14.6.3.2 Design Responsibilities

MSE walls are proprietary wall systems and the structural design of the wall system is provided by the contractor. The structural design of the MSE wall system must include an analysis of internal stability (soil reinforcement pullout and stress) and local stability (facing connection forces and internal panel stresses). Additionally, the contractor should also provide internal drainage. Design drawings and calculations must be submitted to the Structures Design Section for acceptance.

External stability, overall stability and settlement calculations are the responsibility of the WISDOT/Consultant designer. Compound external stability is the responsibility of the Contractor. Soil borings and soil design parameters are provided by Geotechnical Engineer.

Although abutment loads can be supported on spread footings within the reinforced soil zone, it is WisDOT policy to support the abutment loads for multiple span structures on piles or shafts that pass through the reinforced soil zone to the in-situ soil below. Piles shall be driven prior to the placement of the reinforced earth. Strip type reinforcement can be skewed around the piles but must be connected to the wall panels and must extend to the rear of the reinforced soil zone.

For continuous welded wire fabric reinforcement, the contractor should provide details on the plans showing how to place the reinforcement around piles or any other obstacle. Abutments for single span structures may be supported by spread footings placed within the soil reinforcing zone, with WISDOT's approval. Loads from such footings must be considered for both internal wall design and external stability considerations.

14.6.3.3 Design Steps

Design steps specific to MSE walls are described in FHWA publication No*. FHWA-NHI-10-24* and modified shown below:

1. Establish project requirements including all geometry, loading conditions (transient and/or permanent), performance criteria, and construction constraints.
2. Evaluate existing topography, site subsurface conditions, in-situ soil/rock properties, and wall backfill parameters.
3. Select MSE wall using project requirement per step 1 and wall selection criteria discussed in 14.3.1.
4. Based on initial wall geometry, estimate wall embedment depth and length of reinforcement.
5. Estimate unfactored loads including earth pressure for traffic surcharge or sloping back slope and /or front slope.
6. Summarize load factors, load combinations, and resistance factors
7. Calculate factored loads for all appropriate limit states and evaluate (external stability) at Strength I Limit State
   a. sliding
   b. eccentricity
   c. bearing
8. Compute settlement at Service limit states
9. Compute overall stability at Service limit states
10. Compute vertical and lateral movement
11. Design wall surface drainage systems
12. Compute internal stability
    a. Select reinforcement
    b. Estimate critical failure surface
    c. Define unfactored loads
    d. Calculate factored horizontal stress and maximum tension at each reinforcement level
    e. Calculate factored tensile stress in each reinforcement
    f. Check factored reinforcement pullout resistance
    g. Check connection resistance requirements at facing
13. Design facing element
14. Design subsurface drainage

Steps 1-11 are completed by the designer and steps 12-14 are completed by the contractor after letting.

14.6.3.4 Initial Geometry

Figure 14.6-1 provides MSE wall elements and dimensions that should be established before making stability computations for the design of an MSE wall. The height (H) of an MSE wall is measured vertically from the top of the MSE wall to the top of the leveling pad. The length of reinforcement (L) is measured from the back of MSE wall panels. Alternately, the length of reinforcement (L1) is measured from the front face for modular block type MSE walls.

The MSE walls, with panel type facings, generally do not exceed heights of 35 feet, and with modular block type facings, should not exceed heights of 22 feet. Wall heights in excess of these limits will require approval on a case by case basis from the WisDOT.

In general, a minimum reinforcement length of 0.7H or 8 feet whichever is greater shall be provided. MSE wall structures with sloping surcharge fills or other concentrated loads will generally require longer reinforcement lengths of 0.8H to 1.1H. As an exception, a minimum reinforcement length of 6.0 feet or 0.7H may be provided in accordance with **LRFD [C11.10.6.2.1]** provided all conditions for external and internal stability are met and smaller compaction equipment is used on a case by case basis as approved by WisDOT. MSE walls may be built to heights mentioned above; however, the external stability requirements may limit MSE wall height due to bearing capacity, settlement, or stability problems.

14.6.3.4.1 Wall Embedment

The minimum wall embedment depth to the bottom of the MSE wall reinforced backfill zone (top of the leveling pad shown in **LRFD [Figure 11.10.2-1]** and Figure 14.6-1 shall be based on external stability analysis (sliding, bearing resistance, overturning, and settlement) and the global (overall) stability requirements.

Minimum MSE wall leveling pad (and front face) embedment depths below lowest adjacent grade in front of the wall shall be in accordance with **LRFD [11.10.2.2]**, including the minimum embedment depths indicated in **LRFD [Table C11.10.2.2-1]** or 1.5 ft. whichever is greater. The embedment depth of MSE walls along streams and rivers shall be at least 2.0 ft below the potential scour elevation in accordance with **LRFD [11.10.2.2]** and the *Bridge Manual*.

WisDOT policy item:

The minimum depth of embedment of MSE walls shall be 1.5 feet

14.6.3.4.2 Wall Backslopes and Foreslopes

The wall Backslopes and Foreslopes shall be designed in accordance with 14.4.5.4.4. A minimum horizontal bench width of 4 ft (measured from bottom of wall horizontally to the slope face) shall be provided, whenever possible, in front of walls founded on slopes. This minimum bench width is required to protect against local instability near the toe of the wall.

14.6.3.5 External Stability

The external stability of the MSE walls shall be evaluated for sliding, limiting eccentricity, and bearing resistance at the Strength I limit state. The settlement shall be calculated at Service I limit state.

Unfactored loads and factored load shall be developed in accordance with 14.6.3.5.1. It is assumed that the reinforced mass zone acts as a rigid body and that wall facing, the reinforced soil and reinforcement act as a rigid body.

For adequate stability, the goal is to have the factored resistance greater than the factored loads. According to publication FHWA-NHI-10-024, a capacity to demand ratio (CDR) can be used to quantify the factored resistance and factored load. CDR has been used to express the safety of the wall against sliding, limiting eccentricity, and bearing resistance.

14.6.3.5.1 Unfactored and Factored Loads

Unfactored loads and moments are computed based on initial wall geometry and using procedures defined in 14.4.5.4.5. The loading diagrams for one of the 3 possible earth pressure conditions are developed. These include 1) horizontal backslope with traffic surcharge shown in Figure 14.4-2; 2) sloping backslope shown in Figure 14.4-3; and, 3) broken backslope condition as shown in Figure 14.4-4.

The computed nominal loads discussed in 14.5.4 are multiplied by applicable load factors given in Table 14.4-1. A summary of load factors and load combinations as applicable for typical MSE wall stability check is presented in Table 14.6-4. Computed factored load and moments are used for performing stability checks.

14.6.3.5.2 Sliding Stability

The stability should be computed in accordance with **LRFD [11.10.5.3]** and **LRFD [10.6.3.4]**. The sliding stability analysis shall also determine the minimum resistance along the following potential surfaces in the zones shown in **LRFD [Figure 11.10.2.1]**.

- Sliding within the reinforced backfill (performed by contractor)
- Sliding along the reinforced back-fill/base soil interface (performed by designer)

The coefficient of friction angle shall be determined as:

- For discontinuous reinforcements, such as strips – the lesser of friction angle of either reinforced backfill, ϕ_r, the foundation soil, ϕ_{fd}.
- For continuous reinforcements, such as grids and sheets – the lesser of ϕ_r or ϕ_{fd} and ρ.

No passive soil pressure is allowed to resist sliding. The component of the passive resistance shall be ignored due to the possibility that permanent or temporary excavations in front of the wall could occur during the service life of the structure and lead to partial or complete loss of passive resistance. The shear strength of the facing system is also ignored.

For adequate stability, the factored resistance should be greater than the factored load for sliding,

The following equation shall be used for computing sliding:

$$R_\tau = \phi\ R_n = \phi_\tau{*}(V){*}(\tan\delta)$$

Where:

R_R	=	Factored resistance against failure by sliding
R_n	=	Nominal sliding resistance against failure by sliding
R_τ	=	Nominal sliding resistance between soil and foundation
ϕ_τ	=	Resistance factor for shear between the soil and foundation per **LRFD [Table 11.5.6.1]**; 1.0
V	=	Factored vertical dead load
δ	=	Friction angle between foundation and soil
ρ	=	Maximum soil reinforcement interface angle **LRFD [11.11.5.3]**
$\tan\delta$	=	$\tan \phi_{fd}$ where ϕ is lesser of (ϕ_τ , ϕ_{fd} , ρ)
H_{tot}	=	Factored total horizontal load for Strength Ia
CDR	=	$R_\tau/H_{tot} \geq 1$

14.6.3.5.3 Eccentricity Check

The eccentricity check is performed in accordance with **LRFD [11.10.5.5]** and using procedure given in publication, *FHWA-NHI-10-025*

The eccentricity is computed using:

$$e = B/2 - X_0$$

Where:

$$X_0 = \frac{\sum M_V - M_H}{\Sigma V}$$

Where:

ΣM_V = Summation of Resisting moment due to vertical earth pressure

ΣM_H = Summation of Moments due to Horizontal Loads

ΣV = Summation of Vertical Loads

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

$CDR = e_{max}/e > 1$

14.6.3.5.4 Bearing Resistance

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

The bearing resistance computation requires:

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

σ_v = Vertical pressure

ΣV = Sum of all vertical forces

B = Reinforcement length

e = Eccentricity = $B/2 - X_0$

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

ΣM_V = Total resisting moments

ΣM_H = Total driving moments

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

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

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

Where:

q_r = Factored bearing resistance

q_n = Nominal bearing resistance computed using **LRFD [10.6.3.1.2a-1]**

ϕ_b = 0.65 using **LRFD Table [11.5.6.1]**

CDR = q_r/σ_v >1.0

14.6.3.6 Vertical and Lateral Movement

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

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

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

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

14.6.3.7 Overall Stability

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

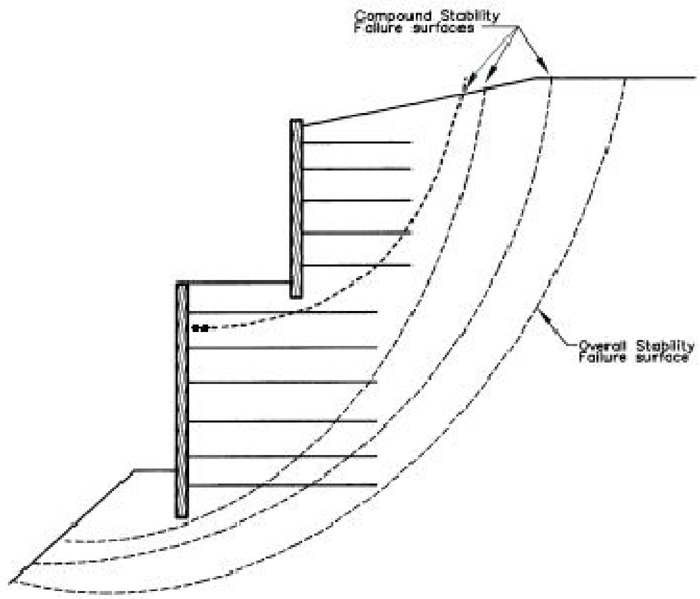


Figure 11.10.4.3-1 Overall and Compound Stability of Complex MSE Wall Systems.

Figure 14.6-4
MSE Walls Overall and Compound Stability
(Source *AASHTO LRFD*)

14.6.3.8 Internal Stability

Internal stability of MSE walls shall be performed by the wall contractor/supplier. The internal stability (safety against structural failure) shall be performed in accordance with **LRFD [11.10.6]** and shall be evaluated with respect to following at the Strength Limit:

- Tensile resistance of reinforcement to prevent breakage of reinforcement
- Pullout resistance of reinforcement to prevent failure by pullout
- Structural resistance of face elements and face elements connections

14.6.3.8.1 Loading

Figure 14.4-11 shows internal failure mechanism of MSE walls due to tensile and pullout failure of the soil reinforcement. The maximum factored tension load (T_{max}) due to tensile and pullout reinforcement shall be computed at each reinforcement level using the *Simplified Method* approach in accordance with **LRFD [11.10.6.2]**. Factored load applied to the reinforcement-facing connection (T_0) shall be equal to maximum factored tension reinforcement load (T_{max}) in accordance with **LRFD [11.10.6.2.2].**

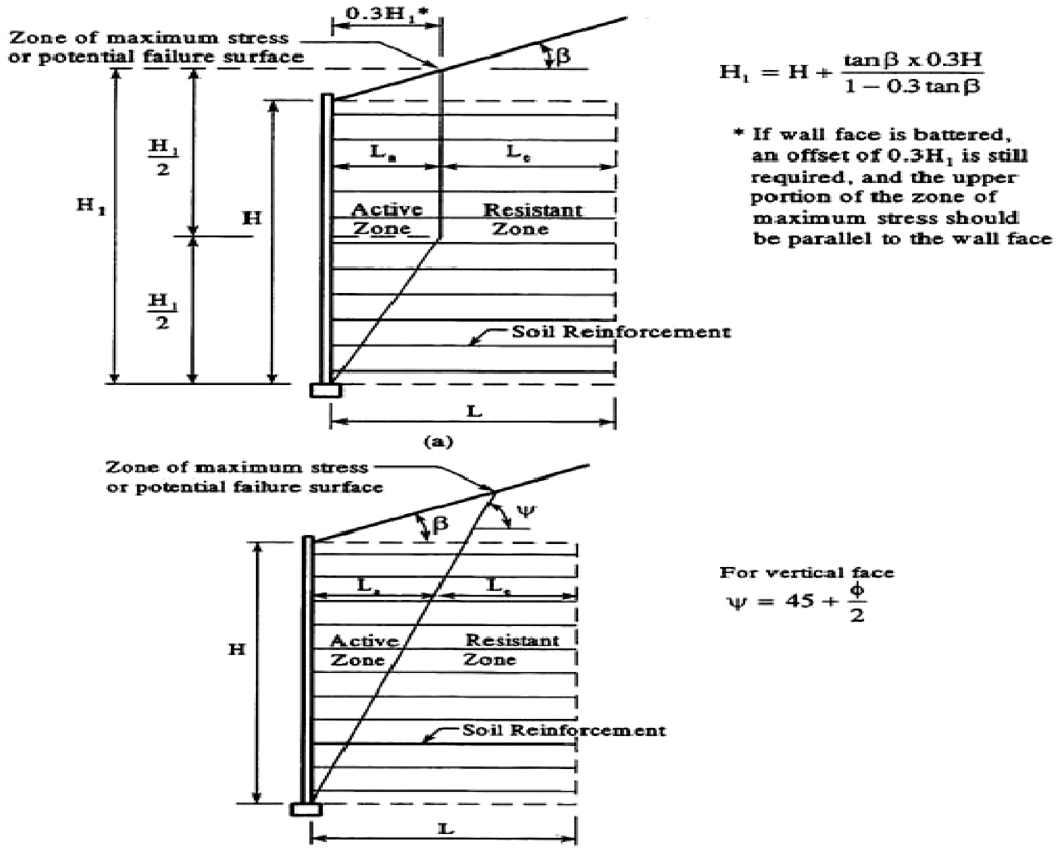
14.6.3.8.2 Reinforcement Selection Criteria

At each reinforcement level, the reinforcement must be sized and spaced to preclude rupture under the stress it is required to carry and to prevent pullout for the soil mass. The process of sizing and designing the reinforcement consists of determining the maximum developed tension loads, their location, along a locus of maximum stress and the resistance provided by reinforcement in pullout capacity and tensile strength.

Soil reinforcements are either extensible or inextensible as discussed in 14.6.2.2.

When inextensible reinforcements are used, the soil deforms more than the reinforcement. The critical failure surface for this reinforcement type is determined by dividing the zone into active and resistant zones with a bilinear failure surface as shown in part (a) of Figure 14.6-5.

When extensible reinforcements are used, the reinforcement deforms more than soil and it is assumed that shear strength is fully mobilized and active earth pressure developed. The critical failure surface for both horizontal and sloping backfill conditions are represented as shown in lower part (b) of Figure 14.6-5.



For walls with a face batter angle (θ) 10° or more from the vertical,

$$\tan(\psi - \theta) = \frac{-\tan(\phi - \beta) + \sqrt{\tan(\phi - \beta)[\tan(\phi - \beta) + \cot(\phi + \theta - 90)][1 + \tan(\delta + 90 - \theta)\cot(\phi + \theta - 90)]}}{1 + \tan(\delta + 90 - \theta)[\tan(\phi - \beta) + \cot(\phi + \theta - 90)]}$$

with $\delta = \beta$

θ = wall batter angle (b)

For wall with a broken backslope, use $\delta = \beta_{eq}$ as shown in Figure 6-20.

Figure 14.6-5
Location of Potential Failure Surface for Internal Stability of MSE Walls
(Source *AASHTO LRFD*)

14.6.3.8.3 Factored Horizontal Stress

The *Simplified Method* is used to compute maximum horizontal stress and is computed using the equation

$$\sigma_H = \gamma_P(\sigma_v k_r + \Delta\sigma_H)$$

Where:

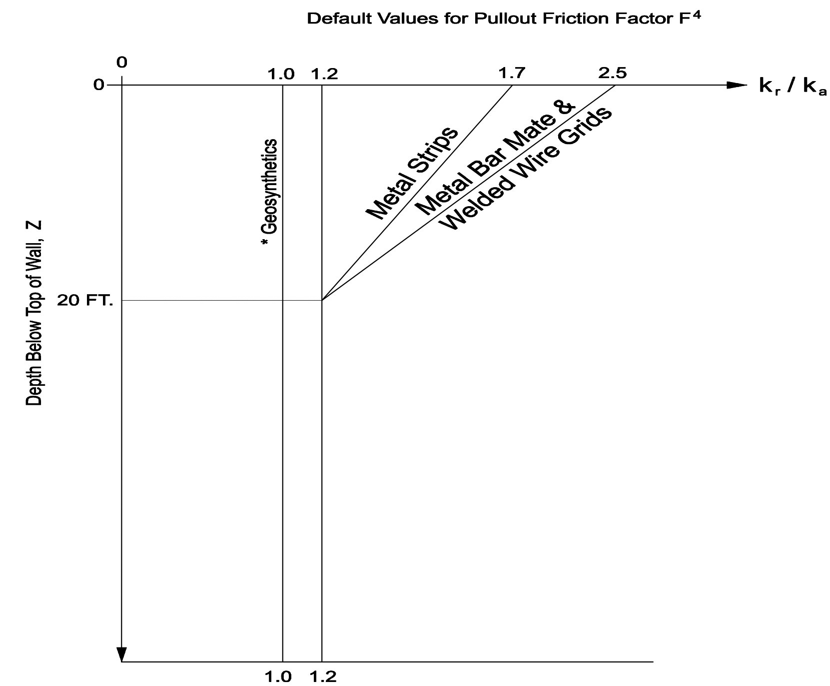
γ_P = Maximum load factor for vertical stress (EV)

k_r = Lateral earth pressure coefficient computed using k_r/k_a

σ_V = Pressure due to reinforce soil mass and any surcharge loads above it

$\Delta\sigma_H$ = Horizontal stress at reinforcement level resulting in a concentrated horizontal surcharge load

Research studies have indicated that the maximum tensile force is primarily related to the type of reinforcement in the MSE mass, which, in turn, is a function of the modulus extensibility, and density of reinforcement. Based on this research, a relationship between the type of reinforcement and the overburden stress has been developed and is shown in Figure 14.6-6.



* Does not apply to polymer strip reinforcement

Figure 14.6-6
Variation of the Coefficient of Lateral Stress Ratio with Depth
(Source *AASHTO LRFD*)

Lateral stress ratio $k_r/k_{a,}$ can be used to compute k_r at each reinforcement level. For vertical face batter <10^0, K_a is obtained using Rankine theory. For wall face with batter greater than 10^0 degrees, Coulomb's formula is used. If present, surcharge load should be added into the estimation of $\sigma_{V.}$. For the simplified method, vertical stress for the maximum reinforcement load calculations are shown in Figure 14.6-7 .

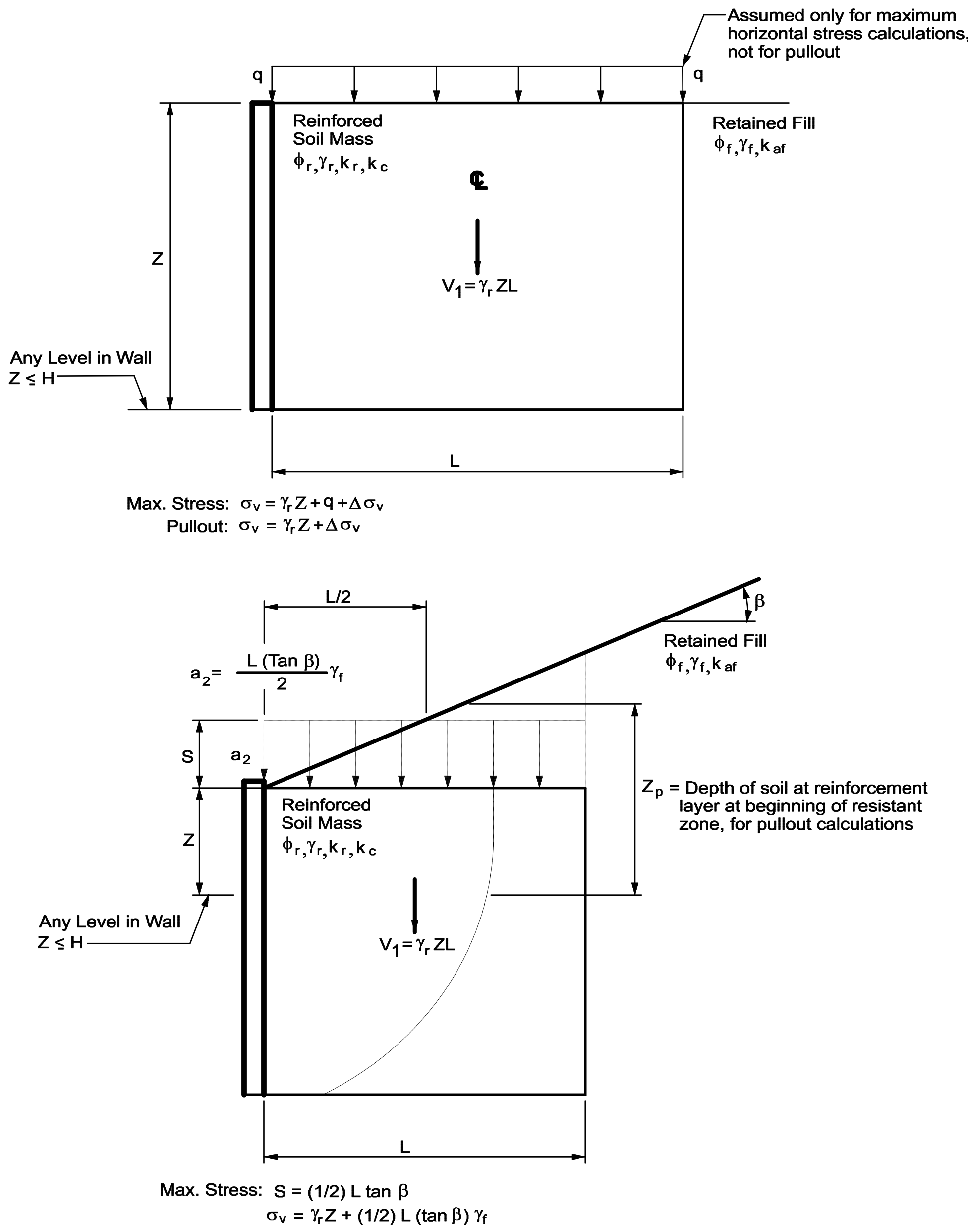


Figure 14.6-7

Calculation of Vertical Stress for Horizontal and Sloping Backslope for Internal Stability (Source *AASHTO LRFD*)

14.6.3.8.4 Maximum Factored Tension Force

The maximum tension load also referred as maximum factored tension force is applied to the reinforcements layer per unit width of wall (T_{max}) will be based on the reinforcement vertical spacing (S_V) as under:

$T_{max} = \sigma_H S_V$

Where:

T_{max}	=	Maximum tension load
σ_H	=	Factored horizontal load defined in 14.6.3.8.3

$T_{max\text{-}UWR}$ may also be computed at each level for discrete reinforcements (metal strips, bar mats, grids, etc) per a defined unit width of reinforcement

$T_{max\text{-}UWR}$	=	$(\sigma_H S_V)/R_C$
R_C	=	Reinforcement coverage ratio **LRFD [11.10.6.4.1]**

14.6.3.8.5 Reinforcement Pullout Resistance

MSE wall reinforcement pullout capacity is calculated in accordance with **LRFD [11.10.6.3]**. The potential failure surface for inextensible and extensible wall system and the active and resistant zones are shown in Figure 14.6-5. The pullout resistance length, L_e, shall be determined using the following equation

$$\phi L_e = \frac{T_{max}}{\left(F^* \cdot \alpha \cdot \sigma'_v \cdot C \cdot R_c\right)}$$

Where:

L_e	=	Length of reinforcement in the resistance zone
T_{max}	=	Maximum tension load
ϕ	=	Resistance factor for reinforcement pullout
F*	=	Pullout friction factor, Figure 14.6-8
α	=	Scale correction factor
σ'_V	=	Unfactored effective vertical stress at the reinforcement level in the resistance zone
C	=	2 for strip, grid, and sheet type reinforcement
R_c	=	Reinforcement coverage ratio **LRFD [11.10.6.4.3.2.1]**

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

Reinforcement Type	α
All steel reinforcement	1.0
Geogrids	0.8
Geotextiles	0.6

Table 14.6-2
Typical values of α
(Source **LRFD [Table 11.10.6.3.2.1]**)

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

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

$$L = L_e + L_a$$

Where:

L_e = Length of reinforcement in the resistance zone

L_a = Remainder length of reinforcement

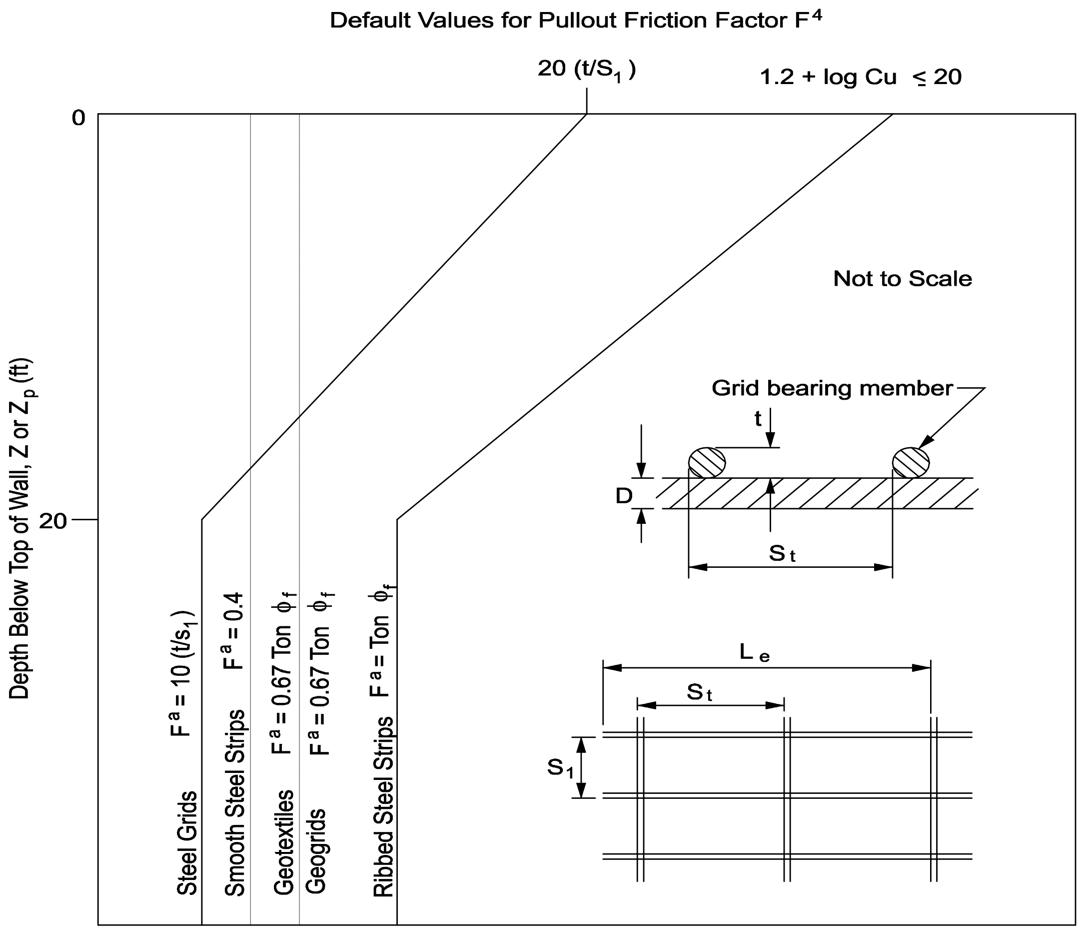


Figure 14.6-8
Typical Values of F*
(Source: **LRFD Figure [11.10.6.3.1]**)

14.6.3.8.6 Reinforced Design Strength

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

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

Where

ϕ = Resistance factor for tensile resistance

R_c = Reinforcement coverage ratio

T_{al} = Nominal tensile resistance (reinforcement design strength) at each reinforcement level

The value for T_{MAX} is calculated with a load factor of 1.35 for vertical earth pressure, EV. The tensile resistance factor for metallic and geosynthetic reinforcement is based on the following:

Metallic Reinforcement	Strip Reinforcement • Static Loading	 0.75
	Grid Reinforcement • Static Loading	 0.65
Geosynthetic Reinforcement	• Static Loading	0.90

Table 14.6-3
Resistance Factor for Tensile and Pullout Resistance
(Source **LRFD Table [11.5.6.1]**)

14.6.3.8.7 Calculate T_{al} for Inextensible Reinforcements

T_{al} for inextensible reinforcements is computed as below:

$T_{al} = (A_c F_y)/b$

Where:

F_y = Minimum yield strength of steel

b = Unit width of sheet grid, bar, or mat

A_c = Design cross sectional area corrected for corrosion loss

14.6.3.8.8 Calculate T_{al} for Extensible Reinforcements

The available long-term strength, T_{al}, for extensible reinforcements is computed as:

$$T_{al} = \frac{T_{ult}}{RF} = \frac{Tult}{RF_{ID} * RF_{CR} * RF_D}$$

Where:

T_{ult} = Minimum average roll value ultimate tensile strength

RF = Combined strength reduction factor to account for potential long term degradation due to installation, damage, creep, and chemical aging

RF_{ID} = Strength Reduction Factor related to installation damage

RF_{CR} = Strength Reduction Factor caused by creep due to long term tensile load

RF_{D} = Strength Reduction Factor due to chemical and biological degradation

RF shall be determined from product specific results as specified in **LRFD [11.10.6.4.3b]**.

14.6.3.8.9 Design Life of Reinforcements

Long term durability of the steel and geosynthetic reinforcement shall be considered in MSE wall design to ensure suitable performance throughout the design life of the structure.

The steel reinforcement shall be designed to achieve a minimum designed life in accordance with **LRFD [11.5.1]** and shall follow the provision of **LRFD [7.6.4.2]**. The provision for corrosion loss shall be considered in accordance with the guidance presented in **LRFD [11.10.6.4.2a]**.

The durability of polymeric reinforcement is influenced by time, temperature, mechanical damage, stress levels, and changes in molecular structure. The strength reduction for geosynthetic reinforcement shall be considered in accordance with **LRFD [11.10.6.4.2b]**.

14.6.3.8.10 Reinforcement /Facing Connection Design Strength

Connections shall be designed to resist stresses resulting from active forces as well as from differential movement between the reinforced backfill and the wall facing elements in accordance with **LRFD [11.10.6.4.4]**.

<u>Steel Reinforcement</u>

Capacity of the connection shall be tested per **LRFD [5.11.3].** Elements of the connection which are embedded in facing elements shall be designed with adequate bond length and bearing area in the concrete, to resist the connection forces. The steel reinforcement connection strength requirement shall be designed in accordance with **LRFD [11.10.6.4.4a]**.

Connections between steel reinforcement and the wall facing units (e.g. bolts and pins) shall be designed in accordance with **LRFD [6.1.3]**. Connection material shall also be designed to accommodate loss due to corrosion.

<u>Geosynthetic Reinforcement</u>

The portion of the connection embedded in the concrete facing shall be designed in accordance with **LRFD [5.11.3]**. The nominal geosynthetic connection strength requirement shall be designed in accordance with **LRFD [11.10.6.4.4b]**.

14.6.3.8.11 Design of Facing Elements

Precast Concrete Panel facing elements are designed to resist the horizontal forces developed internally within the wall. Reinforcement is provided to resist the average loading conditions at each depth in accordance with structural design requirements in *AASHTO LRFD*. The embedment of the reinforcement to panel connector must be developed by test, to ensure that it can resist the maximum tension. The concrete panel must meet temperature and shrinkage steel requirements. Epoxy protection of panel reinforcement is required.

Modular Block Facing elements must be designed to have sufficient inter-unit shear capacity. The maximum spacing between unit reinforcement should be limited to twice the front block width or 2.7 feet, whichever is less. The maximum depth of facing below the bottom reinforcement layer should be limited to the block width of modular facing unit. The top row of reinforcement should be limited to 1.5 times the block width. The factored inter-unit shear capacity as obtained by testing at the appropriate normal load should exceed the factored horizontal earth pressure.

14.6.3.8.12 Corrosion

Corrosion protection is required for all permanent and temporary walls in aggressive environments as defined in **LRFD [11.10.2.3.3].** Aggressive environments in Wisconsin are typically associated with snow removal and areas near storm water pipes in urban areas. MSE walls with steel reinforcement should be protected with a properly designed impervious membrane layer below the pavement and above the first level of the backfill reinforcement. The details of the impervious layer drainage collector pipe can be found in *FHWA-NHI-0043* (FHWA 2001).

14.6.3.9 Wall Internal Drainage

The wall internal drainage should be designed using the guidelines provided in 14.4.7.6. Pipe underdrain must be provided to properly drain MSE walls. Chimney or blanket drains with collector-pipe drains are installed as part of the MSE walls sub-drainage system. Collector pipes with solid pipes are required to carry the discharge away from the wall. All collector pipes and solid pipes should be 6-inch diameter.

14.6.3.10 Traffic Barrier

Design concrete traffic barriers on MSE walls to distribute applied traffic loads in accordance with **LRFD [11.10.10.2]** and WisDOT standard details. Traffic impact loads shall not be transmitted to the MSE wall facing. Additionally, MSE walls shall be isolated from the traffic barrier load. Traffic barrier shall be self-supporting and not rely on the wall facing.

14.6.3.11 Design Example

Example E-2 shows a segmental precast panel MSE wall with steel reinforcement. Example E-3 shows a segmental precast panel MSE wall with geogrid reinforcement. Both design examples include external and internal stability of the walls. The design examples are included in 14.18.

14.6.3.12 Summary of Design Requirements

1. Strength Limit Checks

   a. External Stability

      - Sliding

$$CDR = \left(\frac{R_\tau}{H_{tot}}\right) > 1.0$$

      - Eccentricity Check

$$CDR = \left(\frac{e_{\max}}{e}\right) > 1.0$$

      - Bearing Resistance

$$CDR = \left(\frac{q_r}{\sigma_v}\right) > 1.0$$

   b. Internal stability

      - Tensile Resistance of Reinforcement
      - Pullout Resistance of Reinforcement
      - Structural resistance of face elements and face elements connections

   c. Service Limit Checks

      - Overall Stability
      - Wall Settlement and Lateral Deformation

2. Concrete Panel Facings

   - f'_c = 4000 psi (wet cast concrete)
   - Min. thickness = 5.5 inches
   - Min. reinforcement = 1/8 square inch per foot in each direction (uncoated)

- Min. concrete cover = 1.5 inches
- fy = 60,000 psi

3. Traffic/ Surcharge

- Traffic live load surcharge = 240 lb/ft^2 or
- Non traffic live load surcharge =100 lb/ft^2

4. Reinforced Earthfill

- Unit weight = 120 lb/ft^3
- Angle of internal friction = 30^0, or as determined from Geotechnical analyses (maximum allowed is 36^0)

5. Retained Soil

- Unit weight = 120 lb/ft^3
- Angle of internal friction = 30^0, or as determined from Geotechnical analyses

6. Design Life

- 75 year minimum for permanent walls

7. Soil Pressure Theory

- Coulomb's Theory

8. Soil Reinforcement

For steel or geogrid systems, the minimum soil reinforcement length shall be 70 percent of the wall height and not less than 8 feet. The length of soil reinforcement shall be equal from top to bottom. Soil reinforcement must extend a minimum of 3 feet beyond the failure plane.

9. Summary of Load Combinations and Load Factors

Group	γ_{DC}	γ_{EV}	γ_{LSv}	γ_{LSh}	γ_{EH}	γ_{CT}	Probable use
Strength Ia	0.90	1.00	0.0	1.75	1.50		Sliding, eccentricity
Strength Ib	1.25	1.35	1.75	1.75	1.50		Bearing, Wall strength
Extreme IIa	0.90	1.00	-	-	1.00	1.00	Sliding, eccentricity
Extreme IIb	1.25	1.35	-	-	1.00-	1.00	Bearing
Service I	1.00	1.00	1.00	1.00	1.00	-	Global, settlement, wall crack control

Table 14.6-4
Load Factor Summary for MSE-External Stability
(Source **LRFD [Table 3.4.1]**)

10. Resistance Factors for External Stability

Stability mode	Condition	Resistance Factor
Sliding		1.00
Bearing		0.65
Overall stability	Geotechnical parameters are well defined and slope does not support a structural element	0.75
	Geotechnical parameters are based on limited information, or the slope supports a structural element	0.65

Table 14.6-5
Resistance Factor Summary for MSE-External Stability
(Source **LRFD [Table 11.5.6.1]**)

14.7 Modular Block Gravity Walls

The proprietary modular blocks used in combination with soil reinforcement "Mechanically Stabilized Earth Retaining Walls with Modular Block Facings" can also be used as pure gravity walls (no soil reinforcement). These walls consist of a single row of dry stacked blocks (without mortar) to resist external pressures. These walls can be formed to a tight radius of curvature of 50 ft. or greater. A drawback is that these walls are settlement sensitive. This wall type should only be considered when adequate provisions are taken to keep the surface water runoff and the ground water seepage away from the wall face.

The material specifications for the blocks used for gravity walls are identical to those for the blocks used for block MSE walls as discussed in 14.6.2.3. The modular block gravity walls are proprietary. The wall supplier is responsible for the design of these walls. Design drawings and calculations must be submitted to WisDOT for approval.

The height to which they can be constructed, is a function of the depth of the blocks, the setback of the blocks, the front slope and backslope angle, the surcharge on the retained soil and the angles of internal friction of the retained soil behind the wall. Walls of this type are limited to a height from top of leveling pad to top of wall of 8 feet or less, and are limited to a maximum differential settlement of 1/200.

Footings for modular block gravity walls are either base aggregate dense 1-¼ inch (Section 305 of the *Standard Specifications)* or Grade A concrete. Minimum footing thickness is 12 inches for aggregate and 6 inches for concrete. The width of the footing equals the width of the bottom block plus 12 inches for aggregate footings and plus 6 inches for concrete footings. The bottom modular block is central on the leveling pad. The standard special provisions for Modular Block Gravity Walls require a concrete footing if any portion of a wall is over 5 feet measured from the top of the footing to the bottom of the wall cap.

The coarse aggregate No. 1 (501.2.5.4 of the *Standard Specifications*), is placed within 1 foot behind the back face of the wall, extending down to the bottom of the footing.

14.7.1 Design Procedure for Modular Block Gravity Walls

All modular block gravity walls shall be designed to resist external pressure caused by the supported earth, surcharge loads, and water in accordance with the design criteria discussed in **LRFD [11.11.4]** and 14.4. The design requires an external stability evaluation including sliding, eccentricity check, and bearing resistance check at the Strength I limit state and the evaluation of wall settlement and overall stability at the Service I limit state.

The design of modular block gravity walls provided by the contractor must be in compliance with the WisDOT special provisions for the project and the policy and procedures as stated in 14.15.2 and 14.16. The design must include an analysis of external stability including sliding, eccentricity, and bearing stress check. Horizontal shear capacity between blocks must also be verified by the contractor.

Settlement and overall stability calculations are the responsibility of the designer. The soil design parameters and allowable bearing capacity for the design are provided by the Geotechnical Engineer, including the minimum required block depth.

14.7.1.1 Initial Sizing and Wall Embedment

The minimum embedment to the top of the footing for modular block gravity walls is the same as stated in **LRFD [11.10.2.2]** for mechanically stabilized earth walls. Wall backfill slope shall not be steeper than 2:1. Where practical, a minimum 4.0 ft wide horizontal bench shall be provided in front of the walls.

Wall embedment for prefabricated modular walls shall meet the requirements discussed in section14.4.7.5. The minimum embedment shall be 1.5 ft. or the requirement of scouring or erosion due to flooding defined in 14.6.3.4.1.

14.7.1.2 External Stability

The external stability analyses shall develop the unfactored and factored loads and include evaluations for sliding, eccentricity check, and bearing resistance in accordance with **LRFD [11.11.4]. LRFD [11.11.4.1]** requires that wall stability be performed at every block level.

14.7.1.2.1 Unfactored and Factored Loads

Unfactored loads and moments shall be computed after establishing the initial wall geometry and using procedures defined in 14.4.5.4.5. A load diagram as shown in Figure 14.4-5 shall be developed. Factored loads and moments shall be computed as discussed in 14.4.6 by multiplying applicable load factors given in Table 14.4-1. A summary of load factors and load combinations as applicable for a typical modular block wall is presented in Table 14.7-1. Computed factored load and moments are used for performing stability checks.

14.7.1.2.2 Sliding Stability

Sliding should be considered for the full height wall and at each block level in the wall. The stability should be computed in accordance with **LRFD [10.6.3.4]**, using the following equation:

$$R_R = \phi\, R_n = \phi_\tau\, R_\tau$$

Where:

R_R = Factored resistance against failure by sliding

R_n = Nominal sliding resistance against failure by sliding

ϕ_τ = Resistance factor for shear between soil and foundation per **LRFD [Table 10.5.5.2.2.1]**

ϕ_τ = 0.9 for concrete on sand and 1.0 for soil on soil

R_τ = Nominal sliding resistance between soil and foundation

No passive soil pressure is allowed to resist sliding. The component of the passive resistance shall be ignored due to the possibility that permanent or temporary excavations in front of the wall could occur during the service life of the structure and lead to partial or complete loss of passive resistance.

Interface sliding resistance between concrete blocks shall be calculated using the corrected wall weight based on the calculated hinge height in accordance with **LRFD Figure [11.10.6.4.4b-1]**. Interface friction resistance parameters shall be based on NCMA method. Shear between the blocks must be resisted by friction, keys or pins.

14.7.1.2.3 Bearing Resistance

The bearing resistance of the walls shall be computed in accordance with **LRFD [10.6.3.1]**.

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

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

$q_r = \phi_b q_n \geq \sigma_v$

Where:

q_n	=	Nominal bearing resistance computed using **LRFD [10.6.3.1.2-a]**
$\sum V$	=	Summation of Vertical loads
B	=	Base width
e	=	Eccentricity
ϕ_b	=	0.55 **LRFD Table [11.5.6-1]**

14.7.1.2.4 Eccentricity Check

The eccentricity check shall be performed in accordance with **LRFD [11.11.4.4].** The location of the resultant force should be within the middle half of the base width (e< B/4) for footings on soil, and within (3B/8) for footings on rock.

14.7.1.3 Settlement

The vertical and lateral displacements of prefabricated modular retaining walls must be evaluated for all applicable dead and live load combinations at Service I limit states using procedures described in14.4.7.2 and compared with tolerable movement criteria presented in 14.4.7.2.1. In general, lateral movements of walls on shallow foundations can be estimated assuming the wall rotates or translates as a rigid body due to the effects of earth loads and differential settlements along the base of the wall.

14.7.1.4 Overall Stability

The overall (global) stability shall be evaluated in accordance with **LRFD [11.6.2.3]** and in accordance with 14.4.7.3, with the exception that the entire mass of the modular walls (or the "foundation load"), may be assumed to contribute to the overall stability of the slope. The overall stability check shall be performed by the Geotechnical Unit or Consultant of record.

14.7.1.5 Summary of Design Requirements

1. Stability Evaluations
    - External Stability
        - Eccentricity Check
        - Bearing Check
        - Sliding
    - Settlement
    - Overall/Global
2. Block Data
    - One piece block
    - Minimum thickness of front face = 4 inches
    - Minimum thickness of internal cavity walls other than front face = 2 inches
    - 28 day concrete strength = 5000 psi
    - Maximum water absorption rate by weight = 5%
3. Traffic Surcharge
    - Traffic live load surcharge = 240 lb/ft^2
    - If no traffic live load is present, use 100 lb/ft^2 live load for construction equipment
4. Retained Soil
    - Unit weight γ_f = 120 lb/ft^3
    - Angle of internal friction as determined by Geotechnical Engineer

5. Soil Pressure Theory

   - Use Coulomb Theory

6. Maximum Height = 8 ft.

   (This height is measured from top of leveling pad to bottom of cap. It is not the exposed height). In addition this maximum height may be reduced if there is sloping backfill or a sloping surface in front of the wall.)

7. Load Factors

Group	γ_{DC}	γ_{EV}	γ_{LSv}	γ_{LSh}	γ_{EH}	γ_{CT}	Probable use
Strength Ia	0.90	1.00	0.0	1.75	1.50		Sliding, eccentricity
Strength Ib	1.25	1.35	1.75	1.75	1.50		Bearing /wall strength
Service I	1.00	1.00	1.00	1.00	1.00		Global/settlement/wall crack control

Table 14.7-1
Load Factor Summary for Prefabricated Modular Walls

8. Sliding Resistance Factors

   ϕ_τ = 1.0 **LRFD [Table11.5.6-1]**

9. Bearing Resistance Factors

   ϕ_b = 0.55 **LRFD [Table11.5.6-1]**

14.8 Prefabricated Modular Walls

Prefabricated modular walls systems use interconnected structural elements, which use selected in-fill soil or rock fill to resist external pressures by acting as gravity retaining walls. Metal and precast concrete or metal bin walls, crib walls, and gabion walls are considered under the category of prefabricated modular walls. These walls consist of modular elements which are proprietary. The design of these wall systems is provided by the contractor/wall supplier.

Prefabricated modular walls can be used where reinforced concrete walls are considered. Steel modular systems should not be used where aggressive environmental condition including the use of deicing salts or other similar chemicals are used that may corrode steel members and shorten the life of modular wall systems.

14.8.1 Metal and Precast Bin Walls

Metal bin walls generally consist of sturdy, lightweight, modular steel members called as stringers and spacers. The stringers constitute the front and back face of the bin and spacers its sides. The wall is erected by bolting the steel members together. The flexibility of the steel structure allows the wall to flex against minor ground movement. Metal bin walls are subject to corrosion damage from exposure to water, seepage and deicing salts. To improve the service life of metal bin walls, consideration should be given towards increasing the galvanizing requirements and establishing electrochemical requirements for the confined backfill.

Precast concrete bin walls are typically rectangular interlocking prefabricated concrete modules. A common concrete module typically has a face height varying from 4 to 5 feet, a face length up to 8 feet, and a width ranging from 4 to 20 feet. The wall can be assembled vertically or provided with a batter. A variety of surface treatment can be provided to meet aesthetic requirements. A parapet wall can be provided at the top of the wall and held rigidly by a cast in place concrete slab. A reinforced cast-in-place or precast concrete footing is usually placed at the toe and heel of the wall.

Bin walls are not recommended for applications that require a radius of curvature less than 800 ft. The wall face batter shall not be steeper than 10° or 6:1 (V:H). The base width of bin walls is generally 60% of the wall height. Further description and method of construction can be found in FHWA's publication *Earth Retaining Structures* 2008.

14.8.2 Crib Walls

Crib walls are built using prefabricated units which are stacked and interlocked and filled with free draining material. Cribs consist of solid interlocking reinforced concrete members called rails and tiebacks (sometimes called stretchers and headers). The rails run parallel with the wall face at both the front and rear of the cribbing and the tiebacks run transverse to the rails to tie the structure together. Rails and cross sections of tiebacks form the front face of the wall.

The wall face can either be opened or closed. In closed faced cribs, stretchers are placed in contact with each other. In open face cribs, the stretchers are placed at an interval such that the infill material does not escape through the face. The wall face batter for crib walls shall be no steeper than 4:1.

14.8.3 Gabion Walls

The gabion walls are composed of orthogonal wire cages or baskets tied together and filled with rock fragments. These wire baskets are also known as gabion baskets. The basket size can be varied to suit the terrain with a standard width of 3 feet to standard length varying 3 to 12 feet. The height of these baskets may vary from 1½ feet to 3 feet. Individual wire baskets are filled with rock fragments ranging in size from 4 to 10 inches. After the baskets are filled, the lids are closed and wired shut to form a relatively rigid block. Succeeding rows of the gabions are laced in the field to the underlying gabions and are filled in the same manner until the wall reaches its design height. The rock filled baskets are closed with lids.

The durability of a gabion wall is dependent upon maintaining the integrity of the gabion baskets. Galvanized steel wire is required for all gabion installations. Although gabions are manufactured from a heavy gage wire, there is a potential for damage due to vandalism. While no known case of such vandalism has occurred on any existing WisDOT gabion walls, the potential for such action should be considered at specific sites.

A height of about 18 feet should be considered as a practical limit for gabion walls. Gabion walls have shown good economy for low to moderate heights but lose this economy as height increases. The front and rear face of the wall may be vertical or stepped. A batter is provided for walls exceeding heights of 10 feet, to improve stability. The wall face step shall not be steeper than 6" or 10:1(V:H). The minimum embedment for gabion walls is 1.5 feet. The ratio of the base width to height will normally range from 0.5 to 0.75 depending on backslope, surcharge and angle of internal friction of retained soil. Gabion walls should be designed in cross section with a horizontal base and a setback of 4 to 6 inches at each basket layer. This setback is an aid to construction and presents a more pleasing appearance. The use of a tipped wall base should not be allowed except in special circumstances.

14.8.4 Design Procedure

All prefabricated modular wall systems shall be designed to resist external pressure caused by the supported earth, surcharge loads, and water in accordance with design criteria discussed in **LRFD [11.11.4]** and 14.4 of this chapter. The design requires an external stability evaluation by the WISDOT/Consultant designer, including sliding, eccentricity, and bearing resistance check at the Strength I limit state and the evaluation of wall settlement and overall stability at the Service I limit state.

In addition, the structures modules of the bin and crib walls shall be designed to provide adequate resistance against structural failure as part of the internal stability evaluations in accordance with the guidelines presented in **LRFD [11.11.5]**.

No separate guidance is provided in the *AASHTO LRFD* for the gabion walls, therefore, gabion walls shall be evaluated for the external stability at Strength I and the settlement and overall stability checks at Service I using similar process as that of a prefabricated modular walls.

Since structure modules of the prefabricated modular walls are proprietary, the contractor/ supplier is responsible for the internal stability evaluation and the structural design of the structural modules. The design by contractor shall also meet the requirements for any special provisions. The external stability, overall stability check and the settlement evaluation will be performed by Geotechnical Engineer.

14.8.4.1 Initial Sizing and Wall Embedment

Wall backfill shall not be steeper than 2:1(V:H). Where practical, a minimum 4.0 feet wide horizontal bench shall be provided in front of the walls. A base width of 0.4 to 0.5 of the wall height can be considered initially for walls with no surcharge. For walls with surcharge loads or larger backslopes, an initial base width of 0.6 to 0.7 times can be considered.

Wall embedment for prefabricated modular walls shall meet the requirements discussed in 14.4.7.5. A minimum embedment shall be 1.5 ft or the requirement for scouring or erosion due to flooding.

14.8.5 Stability checks

Stability computations for crib, bin, and gabion modular wall systems shall be made by assuming that the wall modules and wall acts as a rigid body. Stability of gabion walls shall be performed assuming that gabions are flexible.

14.8.5.1 Unfactored and Factored Loads

All modular walls shall be investigated for lateral earth and water pressure including any live and/or dead load surcharge. Dead load due to self weight and soil or rock in-fill shall also be included in computing the unfactored loads. Material properties for selected backfill, concrete, and steel shall be in accordance with guidelines suggested in 14.4.6. The properties of prefabricated modules shall be based on the type of wall modules being supplied by the wall suppliers.

The angle of friction δ between the back of the modules and backfill shall be used in accordance with the **LRFD [3.11.5.9]** and **LRFD [Table C3.11.5.9.1]**. Loading and earth pressure distribution diagram shall be developed as shown in Figure 14.4-6 or Figure 14.4-7

Since infill material and backfill materials of the gabion walls are well drained, no hydrostatic pressure is considered for the gabion walls. The unit weight of the rock-filled gabion baskets shall be computed in accordance with following:

$$\gamma_g = (1-\eta_r)G_s\gamma_w$$

Where:

η_r = Porosity of the rock fill

Gs = Specific gravity of the rock

γ_w = Unit weight of water

Free-draining granular material shall be used as backfill material behind the prefabricated modules in a zone of 1:1 from the heel of the wall. The soil design parameters shall be provided by the Geotechnical Engineer.

Factored loads and moments shall be computed as discussed in 14.4.5.5 and shall be multiplied by applicable load factors given in Table 14.4-1. A summary of load factors and load combinations as applicable for a typical modular block wall is presented in Table 14.8-1

14.8.5.2 External Stability

The external stability of the prefabricated modular walls shall be evaluated for sliding, eccentricity check, and bearing resistance in accordance with **LRFD [11.11.4].** It is assumed that the wall acts as a rigid body. **LRFD [11.11.4.1]** requires that wall stability be performed at every module level. The stability can be evaluated using procedure described in 14.7.1.2.

For prefabricated modular walls, the sliding analysis shall be performed by assuming that 80% of the weight of the soil in the modules is transferred to the footing supports with the remaining soil, weight being transferred to the area of the wall between footings.

The load resisting overturning shall also be limited to 80%, because the interior of soil can move with respect to the retaining module.

The bearing resistance shall be evaluated by assuming that 80% weight of the infill soil is transferred to point (or line) supports at the front or rear of the module.

14.8.5.3 Settlement

The vertical and lateral displacements of prefabricated modular retaining walls must be evaluated for all applicable dead and live load combinations at Service I using procedure described in 14.4.7.2 and compared with tolerable movement criteria presented in 14.4.7.2.1. In general, lateral movements of walls on shallow foundations can be estimated assuming the wall rotates or translates as a rigid body due to the effects of earth loads and differential settlements along the base of the wall.

14.8.5.4 Overall Stability

The overall (global) stability shall be evaluated in accordance with **LRFD [11.6.2.3]** and in accordance with 14.4.7.3 with the exception that the entire mass of the modular walls (or the "foundation load"), may be assumed to contribute to the overall stability of the slope. The overall stability check shall be performed by the Geotechnical Engineer.

14.8.5.5 Structural Resistance

Structural design of the modular units or members shall be performed in accordance with **LRFD [11.11.5]**. The design shall be performed using the factored loads developed for the geotechnical design (external stability) and for the factored pressures developed inside the modules in accordance with **LRFD [11.11.5.1]**. Design shall consider any potential failure mode, including tension, compression, shear, bending, and torsion. The contractor/wall supplier is responsible for the structural design of wall components.

14.8.6 Summary of Design Safety Factors and Requirements

Requirements

Stability Checks

- External Stability
  - Sliding
  - Overturning (eccentricity check)
  - Bearing Stress
- Internal Stability
  - Structural Components
- Settlement
- Overall Stability

Foundation Design Parameters

- Use values provided by Geotechnical Engineer

Concrete and steel Design Data

- f'_c = 4000 psi (or as required by design)
- f_y = 60,000 psi

Use uncoated bars or welded wire fabric

Traffic Surcharge

- Traffic live load surcharge = 240 lb/ft^2
- If no traffic live load is present, use 100 lb/ft^2 live load for construction equipment

Retained Soil

- Unit weight = 120 lb/ft^3

- Angle of internal friction =
    - Use value provided by Geotechnical Engineer
- Rock-infill unit weight =
    - Based on porosity and rock type

Soil Pressure Theory

- Coulomb's Theory for prefabricated wall systems
- Rankine theory or Coulomb theory, at the discretion of designer for gabion walls

7 Load Factors

Group	γ_{DC}	γ_{EV}	γ_{LSv}	γ_{LSh}	γ_{EH}	γ_{ES}	Probable use
Strength Ia	0.90	1.00	0.0	1.75	1.50	1.50	Sliding, eccentricity
Strength Ib	1.25	1.35	1.75	1.75	1.50	1.50	Bearing /wall strength
Service I	1.00	1.00	1.00	1.00	1.00	----	Global/settlement/wall crack control

Table 14.8-1
Load Factor Summary for Prefabricated Modular Walls

14.9 Soil Nail Walls

Soil nail walls consist of installing reinforcement of the ground behind an excavation face, by drilling and installing closely-spaced rows of grouted steel bars (i.e., soil nails). The soil nails are subsequently covered with a facing; used to stabilize the exposed excavation face, support the sub-drainage system (i.e., composite strip drain, collector and drainage pipes), and distribute the soil nail bearing plate load over a larger area. When used for permanent applications, a permanent facing layer, meeting the aesthetic and structural requirement is constructed directly over the temporary facing.

Soil nail walls are typically used to stabilize excavation during construction. Soil nail walls have been used recently with MSE walls to form hybrid wall systems typically known as ‘shored walls’. The soil nails are installed as top down construction. Conventional soil nail wall systems are best suited for sites with dense to very dense, granular soil with some apparent cohesion (sands and gravels), stiff to hard, fine-grained soil (silts and clays) of relatively low plasticity (PI<15), or weak, weathered massive rock with no adversely-oriented planes of weakness. Soil nail wall construction requires that open excavations stand unsupported long enough to allow soil nail drilling and grouting, subdrainage installation, reinforcement, and temporary shotcrete placement. Soil nail walls should not be used below groundwater.

14.9.1 Design Requirements

AASHTO LRFD currently does not include the design and construction of soil nail walls. It is recommended that soil nail walls be designed using methods recommended in *Geotechnical Engineering Circular (GEC) No. 7 – Soil Nail Walls* (FHWA, 2003). The design life of the soil nail walls shall be in accordance with 14.4.3.

The design of the soil nailing walls requires an evaluation of external, internal, and overall stability and facing-connection failure mode as presented in Sections 5.1 thru Sections 5.6 of *(GEC) No. 7 – Soil Nail Walls* (FHWA, 2003).

A permanent wall facing is required for all permanent soil nail walls. Permanent facing is commonly constructed of cast-in-place (CIP) concrete, welded wire mesh (WWM) reinforced concrete and precast fabricated panels. In addition to meeting the aesthetic requirements and providing adequate corrosion protections to the soil nails, design facings for all facing-connection failure modes indicated in FHWA 2003.

Corrosion protection is required for all permanent soil nail wall systems to assure adequate long-tem wall durability. . The level of corrosion protection required should be determined on a project-specific basis based on factors such as wall design life, structure criticality and the electrochemical properties of the supporting soil and rock materials. Criteria for classification of the supporting soil and rock materials as “aggressive” or “non-aggressive” are provided in FHWA 2003.

Soil nails are field tested to verify that nail design loads can be supported without excessive movement and with an adequate margin of safety. Perform both verification and proof testing of designated test nails as recommended in FHWA 2003.

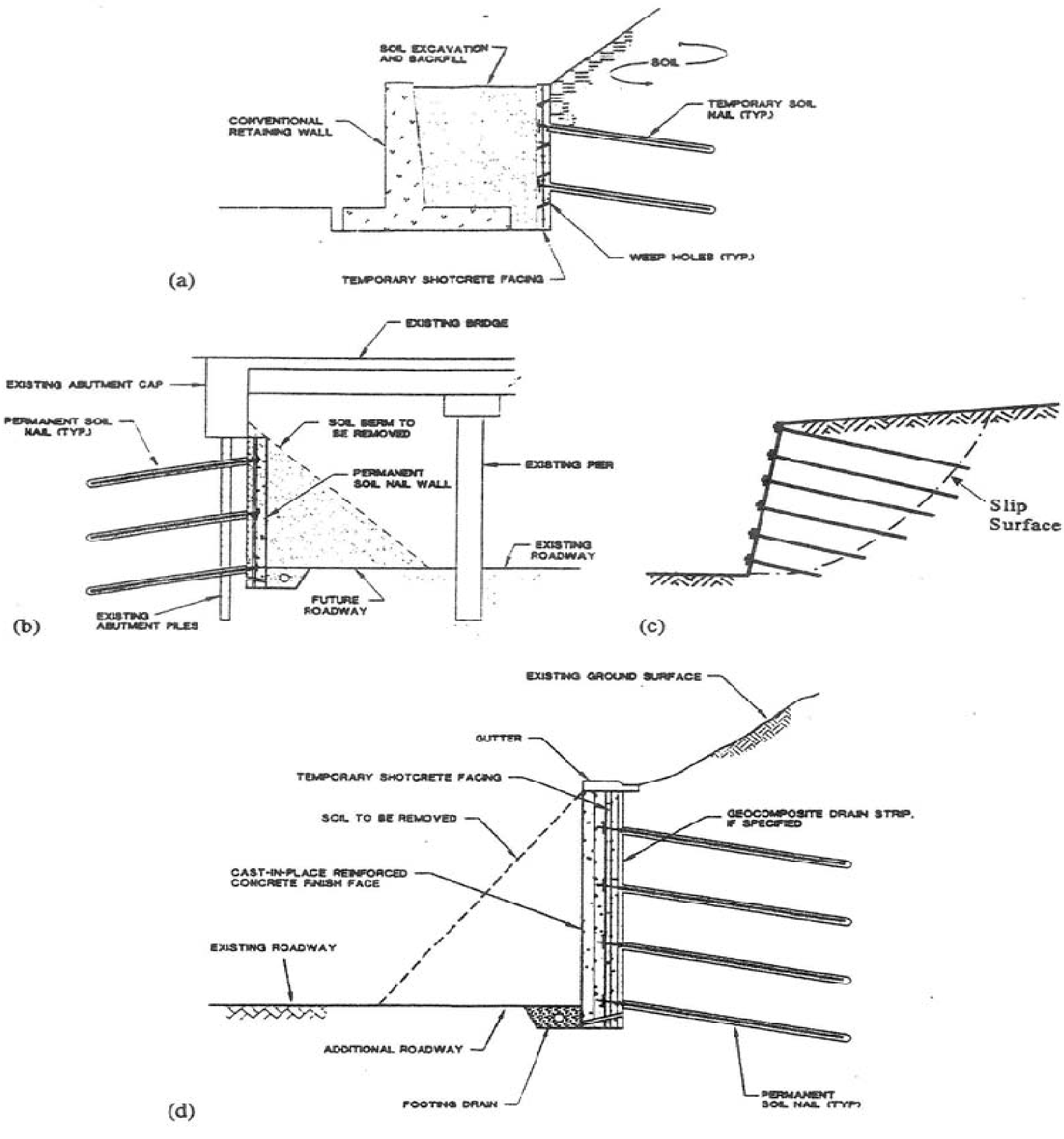


Figure 14.9-1
In-Situ Soil Nailed Walls
(Source: Earth Retaining Structures, 2008)

14.10 Steel Sheet Pile Walls

14.10.1 General

Steel sheet pile walls are a type of non-gravity wall and are typically used as temporary walls, but can also be used for permanent locations.

Sheet piling consists of interlocking steel, precast concrete or wood pile sections driven side by side to form a continuous unit. Steel is used almost exclusively for sheet pile walls. Individual pile sections usually vary from 12 to 21 inches in width, allowing for flexibility and ease of installation. The most common use of sheet piling is for temporary construction of cofferdams, retaining walls or trench shoring. The structural function of sheet piles is to resist lateral pressures due to earth and/or water. The steel manufacturers have excellent design references. Sheet pile walls generally derive their stability from sufficient pile penetration (cantilever walls). When sheet pile walls reach heights in excess of approximately 15 feet, the lateral forces are such that the walls need to be anchored with some form of tieback.

Cofferdams depend on pile penetration, ring action and the tensile strength of the interlocking piles for stability. If a sheet pile cofferdam is to be dewatered, the sheets must extend to a sufficient depth into firm material to prevent a "blow out", that is water coming in from below the base of the excavation. Cross and other bracing rings must be adequate and placed as quickly as excavation permits.

Sheet piling is generally chosen for its efficiency, versatility, and economy. Cofferdam sheet piling and any internal bracing are designed by the Contractor, with the design being accepted by the Department. Other forms of temporary sheet piling are designed by the Department. Temporary sheet piling is not the same as temporary shoring. Temporary shoring is designed by the Contractor and may involve sheet piling or other forms of excavation support.

14.10.2 Sheet Piling Materials

Although sheet piling can be composed of timber or precast concrete members, these material types are seldom, if ever, used on Wisconsin transportation projects.

Steel sheet piles are by far the most extensively used type of sheeting in temporary construction because of their availability, versatility and ability to be reused. Also, they are very adaptable to permanent structures such as bulkheads, seawalls and wharves if properly protected from salt water.

Sheet pile shapes are generally Z, arched or straight webbed. The Z and the medium to high arched sections have high section moduli and can be used for substantial cantilever lengths or relatively high lateral pressures. The shallow arched and straight web sections have high interlocking strength and are employed for cellular cofferdams. The Z-section has a ball-and-socket interlock and the arched and straight webbed sections have a thumb-and-finger interlock capable of swinging 10 degrees. The thumb-and-finger interlock provides high tensile strength and considerable contact surface to prevent water passage. Continuous steel sheet piling is not completely waterproof, but does stop most water from passing through the joints. Steel sheet piling is usually 3/8 to 1/2 inch thick. Designers should specify the required section modulus and embedment depths on the plans, based on bending requirements and also account for corrosion resistance as appropriate.

Refer to steel catalogs for typical sheet pile sections. Contractors are allowed to choose either hot or cold rolled steel sections meeting the specifications. Previously used steel sheet piling may be adequate for some temporary situations, but should not be allowed on permanent applications.

14.10.3 Driving of Sheet Piling

All sheets in a section are generally driven partially to depth before all are driven to the final required depths. There is a tendency for sheet piles to lean in the direction of driving producing a net "gain" over their nominal width. Most of this "gain" can be eliminated if the piles are driven a short distance at a time, say from 6 feet to one third of their length before any single pile is driven to its full length. During driving if some sheet piles strike an obstruction, move to the next pile that can be driven and then return to the piles that resisted driving. With interlock guides on both sides and a heavier hammer, it may be possible to drive the obstructed sheet to the desired depth.

Sheet piles are installed by driving with gravity, steam, air or diesel powered hammers, or by vibration, jacking or jetting depending on the subsurface conditions, and pile type. A vibratory or double acting hammer of moderate size is best for driving sheet piles. For final driving of long heavy piles a single acting hammer may be more effective. A rapid succession of blows is generally more effective when driving in sand and gravel; slower, heavier blows are better for penetrating clay materials. For efficiency and impact distribution, where possible, two sheets are driven together. If sheets adjacent to those being driven tend to move down below the required depth, they are stopped by welding or bolting to the guide wales. When sheet piles are pulled down deeper than necessary by the driving of adjacent piles, it is generally better to fill in with a short length at the top, rather than trying to pull the sheet back up to plan location.

14.10.4 Pulling of Sheet Piling

Vibratory hammers are most effective in removing sheets and typically used. Sheet piles are pulled with air or steam powered extractors or inverted double acting hammers rigged for this application. If piles are difficult to pull, slight driving is effective in breaking them loose. Pulled sheet piling is to be handled carefully since they may be used again; perhaps several times.

14.10.5 Design Procedure for Sheet Piling Walls

A description of sheet pile design is given in **LRFD [11.8.2]** as **"**Cantilevered Wall Design" along with the earth pressure diagrams showing some simplified earth pressures. They are also referred to as flexible cantilevered walls. Steel sheet pile walls can be designed as cantilevered walls up to approximately 15 feet in height. Over 15 feet height, steel sheet pile walls may require tie-backs with either prestressed soil anchors, screw anchors, or deadman-type anchors.

The preferred method of designing cantilever sheet piling is by the "Conventional Method" as described in the *United States Steel Sheet Piling Design Manual* (February,1974). The

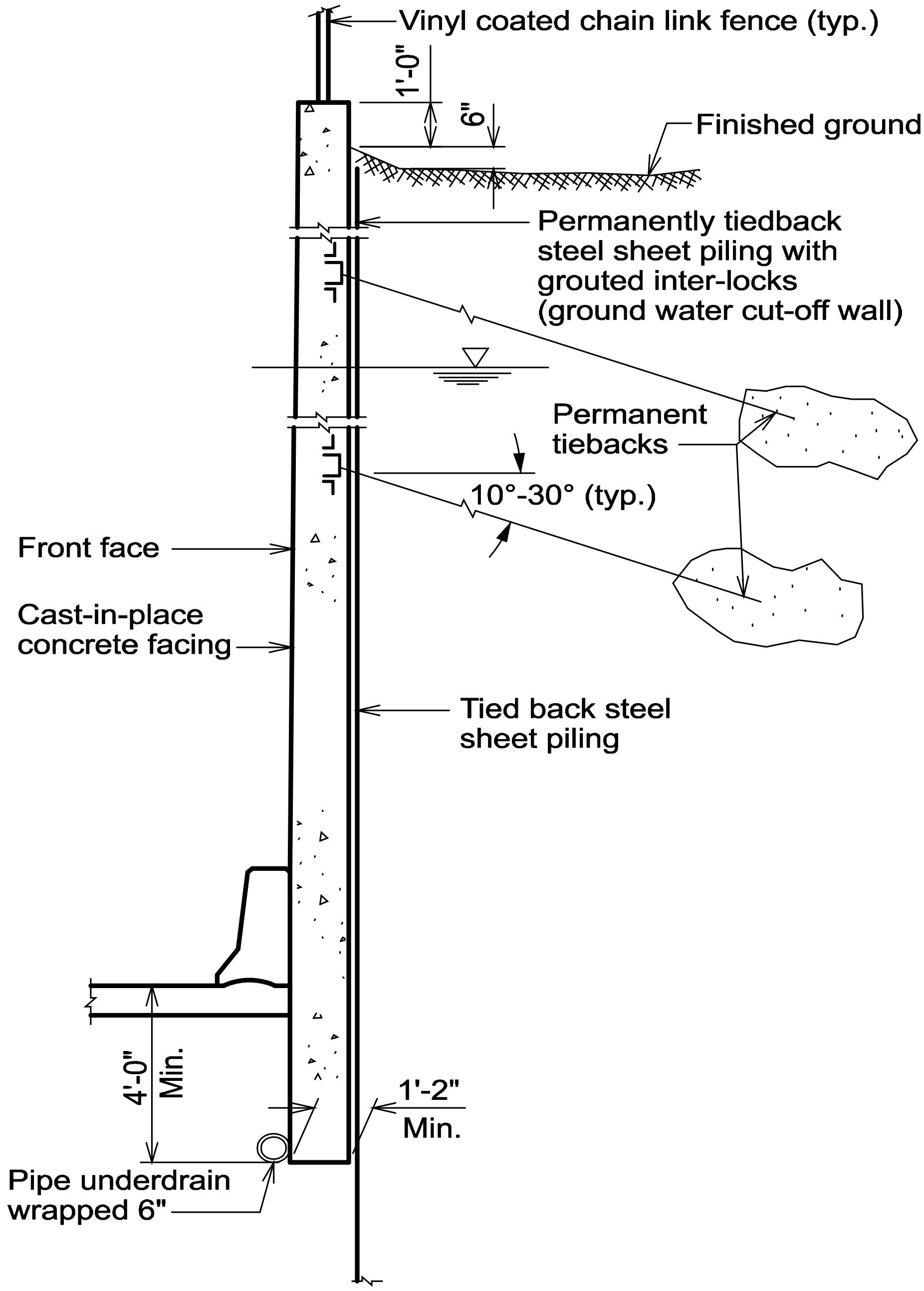
Geotechnical Engineer provides the soil design parameters including cohesion values, angles of internal friction, wall friction angles, soil densities, and water table elevations. The lateral earth pressures for non gravity cantilevered walls are presented in **LRFD [3.11.5.6].**

Anchored wall design must be in accordance with **LRFD [11.5.6]**. Anchors for permanent walls shall be fully encapsulated over their entire length. The anchor hardware shall be designed to have a corrosion resistance durability to ensure a minimum design life of 75 years.

All areas of permanent exposed steel sheet piling above the ground line shall be coated or painted prior to driving, or shall be made from weathering steel. Corrosion potential should be considered in all steel sheet piling designs. Special consideration should be given to permanent steel sheet piling used in areas of northern Wisconsin which are inhabited by corrosion causing bacteria (see *Facilities Development Manual*, Procedure 13-1-15).

Permanent sheet pile walls below the watertable may require the use of composite strip drains, collector and drainage pipes before placement of the final concrete facing.

The appearance of permanent steel sheet piling walls may be enhanced by applying either precast concrete panels or cast-in-place concrete surfacing. Welded stud-shear connectors can be used to attach cast-in-place concrete to the sheet piling. Special surface finishes obtained by using form liners or other means and concrete stain or a combination of stain and paint can be used to enhance the concrete facing aesthetics.



Typical Section Tiedback Retaining Wall

Figure 14.10-1
Typical Anchored Sheet Pile Wall

14.10.6 Summary of Design Requirements

1. Load and Resistance Factor

Load Combination	Load Factors	Resistance Factor
Strength I (maximum)	EH-Horizontal Earth Pressure: δ =1.50 **LRFD [Table 3.4.1-2]**	-----------
Strength I (maximum)	LS-Live Load Surcharge: δ =1.75 **LRFD [Table 3.4.1-1]**	-----------
Strength I (maximum)	-----------	Passive resistance of vertical elements: ϕ=0.75 **LRFD [Table11.5.6-1]**
Service I	-----------	Overall Stability: ϕ=0.75, when geotechnical parameters are well defined, and the slope does not support or contain a structural element
Service I	-----------	Overall Stability: ϕ=0.65, when geotechnical parameters are based on limited information, or the slope does support or contain a structural element

Table 14.10-1
Summary of Design Requirements

2. Foundation design parameters

   Use values provided by the Geotechnical Engineer of record for permanent sheet pile walls. Temporary sheet pile walls are the Contractor's responsibility.

3. Traffic surcharge

   - Traffic live load surcharge = 240 lb/ft^2 or determined by site condition.
   - If no traffic live load is present, use 100 lb/ft^2 live load for construction equipment

4. Retained soil

   - Unit weight = 120 lb/ft^3
   - Angle of internal friction as determined from the Geotechnical Report.

5. Soil pressure theory

   Coulomb Theory.

6. Design life for anchorage hardware

   75 years minimum

7. Steel design properties

   Minimum yield strength = 39,000 psi

14.11 Post and Panel Walls

Post and panel walls are comprised of discrete vertical elements (usually steel H piles) and concrete panels or wood lagging which extend between the vertical elements. The panels are usually constructed of precast reinforced concrete although precast prestressed concrete is also a possibility. Precast prestressed concrete can also be used for the vertical elements. Post and panel walls should be considered if minimum environmental damage and/or disturbances to the site from construction procedures are critical. Post and panel walls may also be used when an irregular rock surface and/or rock near the surface exists at the wall location since the holes for the posts can be drilled into the rock.

14.11.1 Design Procedure for Post and Panel Walls

LRFD [11.8] Non-Gravity Cantilevered Walls covers the design of post and panel walls. A simplified earth pressure distribution diagram is shown in **LRFD [3.11.5.6]** for permanent post and panel walls. Another method that may be used is the "Conventional Method" or "Simplified Method" as described in "*United States Steel Sheet Piling Design Manual*", February, 1974. This method must be modified for the fact that it is based on continuous vertical wall elements whereas, post and panel walls have discrete vertical wall elements. Using "Broms" method for designing drilled shafts is also acceptable. WisDOT can provide the design of these walls.

The maximum spacing between vertical supporting elements (posts) depends on the wall height and the design parameters of the foundation soil. Spacing of 6 to 12 feet is typical. The posts (vertical elements) are set in drilled holes and concrete is placed in the hole after the post is set. The post system must be designed to handle maximum bending moment along length of embedded shaft. The maximum bending moment at any level in the facing can be determined from formulas in **LRFD [11.8.5.1]**. The minimum panel thickness allowed is 6 inches.

The diameter of the drilled shaft is also dependent on the wall height and the design parameters of the foundation soil. The larger the diameter of the drilled shaft the smaller will be the required embedment of the shaft. The designer should try various shaft diameters to optimize the cost of the drilled shaft considering both material cost and drilling costs. Note that drilling costs are a function of both hole diameter and depth.

If the vertical elements are steel they shall be shop painted. Wall panels are usually given a special surface treatment created by brooming or tining vertically, using form liners, or using a pattern of rustication strips. The portion of the panel receiving the special treatment may be recessed, forming a border around the treated area. Concrete paints or stains may be used for color enhancements. When panel heights exceed 15 feet anchored walls may be needed. Anchored wall design must be in accordance with **LRFD [11.9]**. Anchors for permanent walls shall be fully encapsulated over their entire length. The anchor hardware shall be designed to have a corrosion resistance durability to ensure a minimum design life of 75 years.

The concrete for post and panel walls shall have a 28 day compressive strength of 4000 psi if non-prestressed and 5000 psi if prestressed except for the drilled shafts. Concrete for the drilled shafts shall have a 28 day compressive strength of 3500 psi. Reinforcement shall be uncoated Grade 60 in drilled shafts. In lieu of drainage aggregate a membrane may be used to seal the joints between the vertical elements and concrete panels to prevent water leakage. The front face of post and panel walls shall be battered 1/4" per foot to account for short and long term deflection.

14.11.2 Summary of Design Requirements

Requirements

1. Resistance Factors

    - Overall Stability= 0.65 to 0.75 (based on how well defined the geotechnical parameters are and the support of structural elements)
    - Passive Resistance of vertical Elements = 0.75

2. Foundation Design Parameters

    Use values provided by the Geotechnical Engineer

3. Concrete Design Data

    - f'_c = 3500 psi (for drilled shafts)
    - f'_c = 4000 psi (non-prestressed panel)
    - f'_c = 5000 psi (prestressed panel)
    - f_y = 60,000 psi

4. Load Factors

    - Vertical earth pressure = 1.5
    - Lateral earth pressure = 1.5
    - Live load surcharge = 1.75

5. Traffic Surcharge

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

6. Retained Soil

    - Unit weight = 120 lb/ft^3
    - Angle of internal friction - Use value provided by the Geotechnical Engineer

7. Soil Pressure Theory

   Rankine's Theory or Coulombs Theory at the discretion of the designer.

8. Design Life for Anchorage Hardware

   75 year minimum

9. Steel Design Properties (H-piles)

   Minimum yield strength = 50,000 psi

14.12 Temporary Shoring

This information is provided for guidance. Refer to the *Facilities Development Manual* for further details.

Temporary shoring is used to support a temporary excavation or protect existing transportation facilities, utilities, buildings, or other critical features when safe slopes cannot be made for structural excavations. Shoring may be required within the limits of structures or on the approach roadway due to grade changes or staged construction. Temporary shoring generally includes non-anchored temporary sheet piles, temporary post and panel walls, temporary soil nails, cofferdam, or temporary mechanically stabilized earth (MSE) walls.

Temporary shoring is designed by the contractor. Shoring should not be required nor paid for when used primarily for the convenience of the contractor.

14.12.1 When Slopes Won't Work

Typically shoring will be required when safe slopes cannot be made due to geometric constraints of existing and proposed features within the available right-of-way. Occupation and Healthy Safety Administration (OSHA) requirements for temporary excavation slopes vary from a 1H:1V to a 2H:1V. The contractor is responsible for determining and constructing a safe slope based on actual site conditions.

In most cases, the designer can assume that an OSHA safe temporary slope can be cut on a 1.5H:1V slope; however other factors such as soil types, soil moisture, surface drainage, and duration of excavation should also be factored into the actual slope constructed. As an added safety factor, a 3-foot berm should be provided next to critical points or features prior to beginning a 1.5H:1V slope to the plan elevation of the proposed structure. Sufficient room should be provided adjacent to the structure for forming purposes (typically 2-3 feet).

14.12.2 Plan Requirements

Contract plans should schematically show in the plan and profile details all locations where the designer has determined that temporary shoring will be required. The plans should note the estimated length of the shoring as well as the minimum and maximum required height of exposed shoring. These dimensions will be used to calculate the horizontal projected surface area projected on a vertical plane of the exposed shoring face.

14.12.3 Shoring Design/Construction

The Contractor is responsible for design, construction, maintenance, and removal of the temporary shoring system in a safe and controlled manner. The adequacy of the design should be determined by a Wisconsin Professional Engineer knowledgeable of specific site conditions and requirements. The temporary shoring should be designed in accordance with the requirements described in 14.4.2 and 14.4.3. A signed and sealed copy of proposed designs must be submitted to the WisDOT Project Engineer for information.

14.13 Noise Barrier Walls

14.13.1 Wall Contract Process

WisDOT has classified all noise walls (both proprietary and non-proprietary) into three wall systems. All proprietary systems must be pre-approved prior to being considered for use on WisDOT projects. The three noise wall systems that are considered for WisDOT projects include the following:

1. Double-sided sound absorptive noise barriers
2. Single-sided sound absorptive noise barriers
3. Reflective noise barriers

If a wall is required, the designer must determine which wall system or systems are suitable for a given wall location. In some locations all wall systems may be suitable, whereas in other locations some wall systems may not be suitable. Information on aesthetic qualities and special finishes and colors of proprietary systems is available from the manufacturers. Information on approved concrete paints, stains and coatings is also available from the Structures Design Section. Designers are encouraged to contact the Structures Design Section (608-266-8494) if they have any questions about the material presented in the *Bridge Manual*.

The step by step process required to select a suitable wall system or systems for a given wall location is as follows:

Step 1: Investigate alternatives

Investigate alternatives to walls such as berms, plantings, etc.

Step 2: Geotechnical analysis

If a wall is required, geotechnical personnel shall conduct a soil investigation at the wall location and determine soil design parameters for the foundation soil. Geotechnical personnel are also responsible for recommending remedial methods of improving soil bearing capacity if required.

Step 3: Evaluate basic wall restrictions

The designer shall examine the list of suitable wall systems using the Geotechnical Report and remove any system that does not meet usage restrictions for the site.

Step 4: Determine suitable wall systems

The designer shall further examine the list of suitable wall systems for conformance to other considerations. Refer to Chapter 2 – General and Chapter 6 – Plan Preparation for a discussion on aesthetic considerations.

Step 5: Determine contract letting

After the designer has established the suitable wall system(s), the method of contract letting can be determined. The designer has several options based on the contents of the list.

Option 1:

The list contains only non-proprietary systems.

Under Option 1, the designer will furnish a complete design for one of the non-proprietary systems.

Option 2:

The list contains proprietary wall systems only or may contain both proprietary and non-proprietary wall systems, but the proprietary wall systems are deemed more appropriate than the non-proprietary systems.

Under Option 2 the designer will not furnish a design for any wall system. The contractor can build any wall system which is included on the list. The contractor is responsible for providing the complete design of the wall system selected, either by the wall supplier for proprietary walls or by the contractor's engineer for non-proprietary walls. Contract special provisions, if not in the Supplemental Specs., must be included in the contract document for each wall system that is allowed. Under Option 2, at least two and preferably three wall suppliers must have an approved product that can be used at the project site. See the *Facilities Development Manual* (Procedure 19-1-5) for any exceptions.

Option 3:

The list contains proprietary wall systems and non-proprietary wall systems and the non-proprietary systems are deemed equal or more appropriate than the proprietary systems.

Under Option 3 the designer will furnish a complete design for one of the non-proprietary systems, and list the other allowable wall systems.

Step 6: Prepare Contract Plans

Refer to section 14.16 for information required on the contract plans for proprietary systems. If a contractor chooses an alternate wall system, the contractor will provide the plans for the wall system chosen.

Step 7: Prepare Contract Special Provisions

The Structures Design Section and Region Offices have Special Provisions for each wall type and a generic Special Provision to be used for each project. The list of proprietary wall suppliers is maintained by the Materials Quality Assurance Unit.

Complete the generic Special Provision for the project by inserting the list of wall systems allowed and specifying the approved list of suppliers if proprietary wall systems are selected.

Step 8: Submit P.S.& E. (Plans, Specifications and Estimates)

When the plans are completed and all other data is completed, submit the project into the P.S.& E. process. Note that there is one bid item, square feet of exposed wall, for all wall quantities.

Step 9: Preconstruction Review

The contractor must supply the name of the wall system supplier and pertinent construction data to the project manager. This data must be accepted by the Office of Design, Contract Plans Section before construction may begin. Refer to the Construction and Materials Manual for specific details.

Step 10: Project Monitoring

It is the responsibility of the project manager to verify that the project is constructed with the previously accepted contract proposal. Refer to the Construction and Materials Manual for monitoring material certification, construction procedures and material requirements.

14.13.2 Pre-Approval Process

The purpose of the pre-approval process is to ascertain that a particular proprietary wall system has the capability of being designed and built according to the requirements and specifications of WisDOT. Any unique design requirements that may be required for a particular system are also identified during the pre-approval process. A design of a pre-approved system is acceptable for construction only after WisDOT has verified that the design is in accordance with the design procedures and criteria stated in the Certification Method of Acceptance for Noise Barrier Walls.

In addition to design criteria, suppliers must provide materials testing data and certification results for the required tests for durability, etc. The submittal requirements for the pre-approval process and other related information are available from the Materials Quality Assurance Unit, Madison, Wisconsin.

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 532.0201.S.
- Wall Modular Block Mechanically Stabilized Earth LRFD, Item 532.0301.S.
- Wall Concrete Panel Mechanically Stabilized Earth LRFD, Item 532.0501.S
- Wall CIP Facing Mechanically Stabilized Earth LRFD, Item 532.0601.S.
- Wall Wire Faced Mechanically Stabilized Earth LRFD, Item 532.0701.S.
- Wall Gabion LRFD, Item 532.0801.S.
- Wall Modular Bin or Crib LRFD, Item 532.0901.S.
- Temporary Wall Wire Faced Mechanically Stabilized Earth LRFD, Item 532.0751.S.

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

reinforcement elements, procedures for field and laboratory evaluation including instrumentation and special requirements, if any.

5. Sample material and construction control specifications - showing material type, quality, certifications, field testing and placement procedures.

6. A well documented field construction manual describing in detail and with illustrations where necessary, the step by step construction sequence.

7. Details for mounting a concrete traffic barrier on the wall adjoining both concrete and flexible pavements (if applicable).

8. Pullout data for facing block/geogrid connection and soil pullout data (if applicable).

9. Submission of practical application with photos for all materials, surface textures and colors representative of products being certified.

10. Submission, if requested, to an on-site production process control review, and record keeping review.

11. List of installations including owner name and wall location.

12. Limitations of the wall system.

The above materials may be submitted at any time but, to be considered for a particular WisDOT project, must be received a minimum of 15 weeks before the letting date for that project to meet the P.S. & E. schedule. The material should be clearly detailed and presented according to the prescribed outline.

After final review and approval of comments with the Structural Development Section, the manufacturer will be approved to begin presenting the system on qualified projects.

14.16.4 Maintenance of Approval Status as a Manufacturer

The supplier or manufacturer must request to be reapproved bi-annually. The request shall be in writing and certify that the plant production process control and materials testing and design procedures haven't changed since the last review. The request shall be received within two years of the previous approval or the approval status will be terminated. Upon request for re-approval an on-site review of plant process control and materials testing may be conducted by WisDOT personnel. Travel expenses for trips outside the State of Wisconsin involved with this review will be borne by the manufacturer.

For periodic on-site reviews, access to the plant operations and materials records shall be provided to a representative of the Construction Materials Engineer during normal working hours upon request.

If the supplier or manufacturer introduces a new material, or cross-section, or a new design procedure, into its product line, the new feature must be submitted for approval. If the new feature/features are significantly different from the original product, the new product may be subjected to a complete review for approval.

14.16.5 Loss of Approved Status

Approval to deliver the approved system may be withdrawn under the following conditions:

Design Conformance

1. Construction does not follow design procedures.
2. Incorrect design procedures are used on projects.

Materials

3. Inability to consistently supply material meeting specification.
4. Inability to meet test method precision limits for quality control testing.
5. Lack of maintenance of required records.
6. Improper documentation of shipments.
7. Not maintaining an acceptable quality control program.

The decision to remove approval from a manufacturer on a specific system rests with the Construction Materials Engineer for Highways or the State Bridge Engineer.

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

E1: Cast-In-Place Concrete Cantilever Wall on Spread Footing, LRFD

E2: Precast Panel Steel Reinforced MSE Wall, LRFD

E3: Modular Block Facing Geogrid Reinforced MSE Wall, LRFD

E4: Cast-In-Place Concrete Cantilever Wall on Piles, LRFD

E5: Sheet Pile Wall, LRFD

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E14-1 Cast-In-Place Concrete Cantilever Wall on Spread Footing, LRFD

General

This example shows design calculations for a cast-in-place (CIP) concrete wall supported on a spread footing conforming to the LRFD Bridge Design Specifications and the WisDOT Bridge Manual. *(Example is current through LRFD Fifth Edition - 2010)*

Sample design calculations for bearing resistance, external stability (sliding, eccentricity and bearing) and wall strength design will be presented. The overall stability and settlement calculations will not be shown in this example, but are required.

Design steps presented in 14.5.2.1 are used for the wall design.

E14-1.1 Establish Project Requirements

The CIP concrete wall shown in Figure E14-1.1-1 will be designed appropriately to accommodate a State Truck Highway. External stability, overall stability and wall strength are the designer's (WisDOT/Consultant) responsibility.

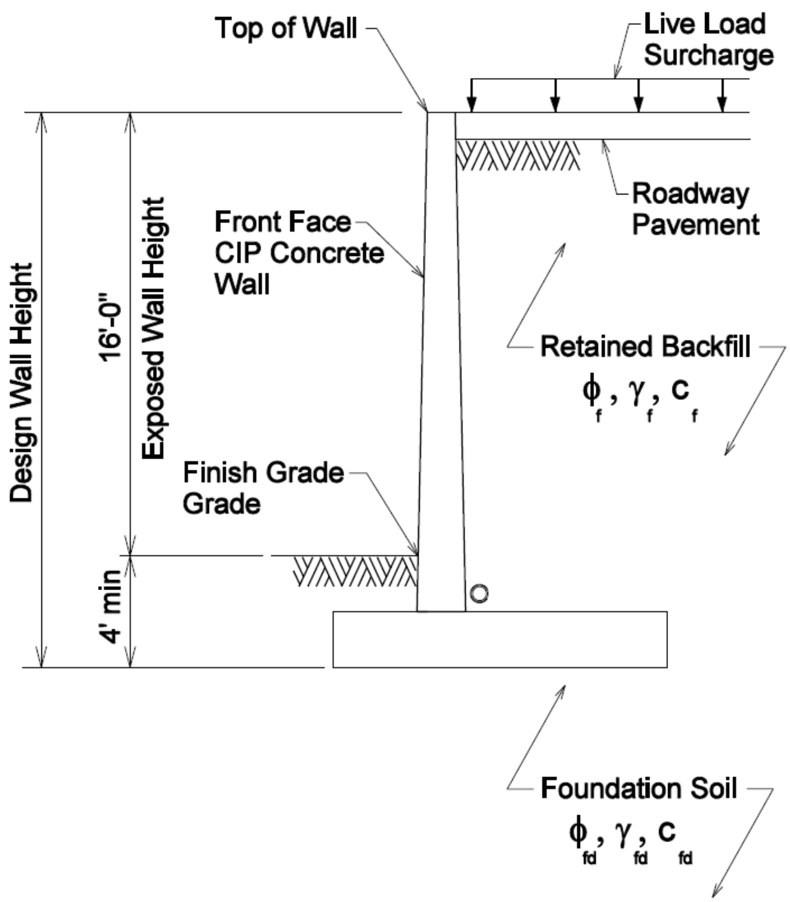


Figure E14-1.1-1
CIP Concrete Wall Adjacent to Highway

E14-1.2 Design Parameters

Project Parameters

$Design\_Life = 75\ years$ Wall design life (min) **LRFD [11.5.1]**

Soil Properties (From Geotechnical Site Investigation Report)

Designer to determine if long-term or short-term soil strength parameters govern external stability.

Backfill Soil Design Parameters

$\phi_f = 30\ deg$ Angle of internal friction

$\gamma_f = 0.120$ Unit weight, kcf

$c_f = 0$ Cohesion, pcf

$\delta = 21\ deg$ Friction angle between fill and wall

Note: Per WisDOT Bridge Manual and Standard Specifications, structural backfill shall be granular and non-expansive.

Foundation Soil Design Parameters

$\phi_{fd} = 34\ deg$ Angle of internal friction

$\gamma_{fd} = 0.120$ Unit of weight, kcf

$c_{fd} = 0$ Cohesion, psf

Reinforced Concrete Parameters

$f'_c = 3.5$ Concrete compressive design strength, ksi (14.5.9)

$\gamma_c = 0.150$ $w_c = \gamma_c$ Unit weight of concrete, ksf

$E_c = 33000\ w_c^{1.5}\sqrt{f'_c}$ Modulus of elasticity of concrete, ksi **LRFD [5.4.2.4]**

$\boxed{E_c = 3587}$ ksi

$f_y = 60$ Yield strength of reinforcing bars, ksi (14.5.9)

$E_s = 29000$ Modulus of elasticity of reinforcing bars, ksi

Live Load Surcharge Parameters

Live load surcharge shall be used when vehicular load is located within H/2 of the backface of the wall **LRFD [3.11.6.4].** The equivalent height of soil for vehicular load, H_{eq}, used for surcharge loads shall be in accordance to **LRFD [Table 3.11.6.4-2]**. However, WisDOT policy for most cases requires an equivalent height of 2.0 feet. The following procedure is used for determining live load surcharge:

$L_{traffic} = 1.0$	Distance from wall backface to edge of traffic, ft
$\frac{H}{2} = 10.00$	Distance from wall backface where live load surcharge shall be considered in the wall design, ft

Note: The wall height used is the exposed height plus an assumed 4 feet embedment ($H=H_e+4$ feet)

Shall live load surcharge be included? check = "YES"

$h_{eq} = 2.0$	Equivalent height of soil for surcharge load, ft (14.4.5.4.2)

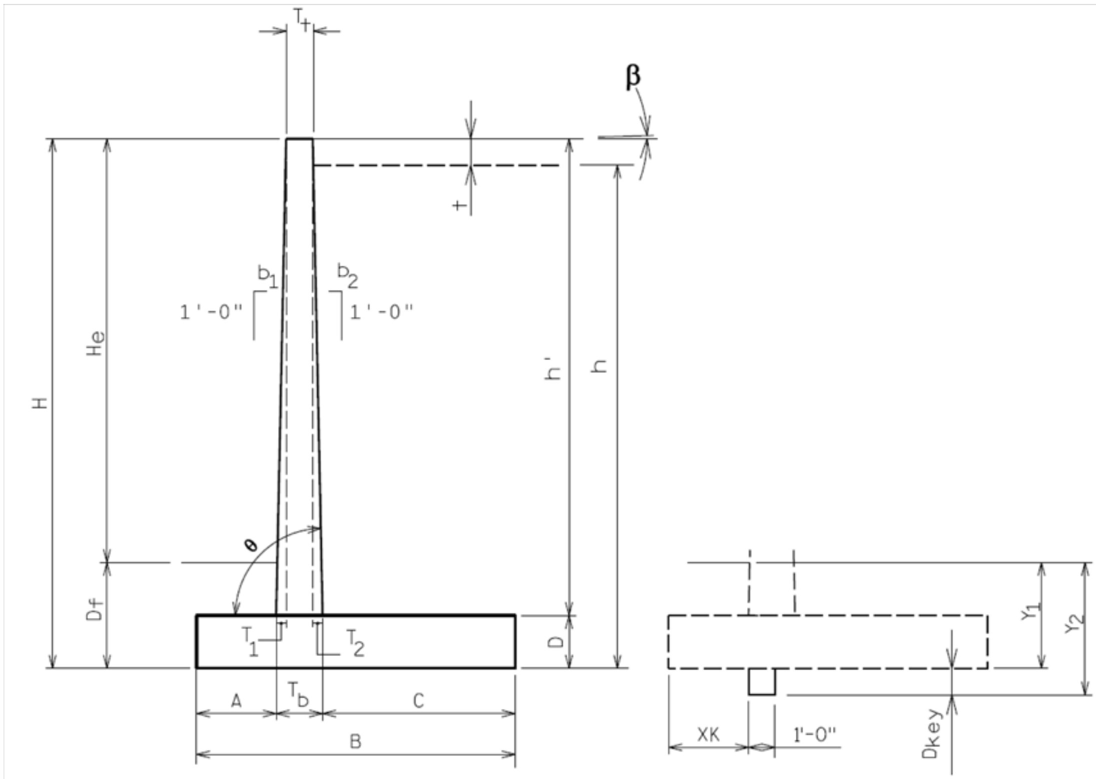
Pavement Parameters

$\gamma_p = 0.150$	Pavement unit weight, kcf

Resistance Factors

$\phi = 1.00$	Sliding resistance
$\phi_b = 0.45$	Bearing resistance (theoretical method, in sand, using SPT) **LRFD [Table 10.5.5.2.2-1]**
$\phi_\tau = 1.00$	Sliding resistance
$\phi_{\tau 1} = 0.80$	Sliding resistance (cast-in-place concrete on sand) **LRFD [Table 10.5.5.2.2-1]**
$\phi_{\tau 2} = 1.00$	Sliding resistance (soil-on-soil)
$\phi_{ep} = 0.50$	Sliding resistance (passive earth pressure component of sliding resistance) **LRFD [Table 10.5.5.2.2-1]**
$\phi_F = 0.90$	Concrete flexural resistance (Assuming tension-controlled) **LRFD [5.5.4.2.1]**
$\phi_V = 0.90$	Concrete shear resistance **LRFD [5.5.4.2.1]**

E14-1.3 Define Wall Geometry



with Shear Key

Figure E14-1.3-1
CIP Concrete Wall Geometry

Wall Geometry

$H_e = 16.0$	Exposed wall height, ft
$D_f = 4.0$	Footing cover, ft (WisDOT policy 4'-0" minimum)
$H = H_e + D_f$	Design wall height, ft
$T_t = 1.0$	Stem thickness at top of wall, ft
$b_1 = 0.25$	Front wall batter, in/ft (b_1H:12V)
$b_2 = 0.50$	Back wall batter, in/ft (b_2H:12V)
$\beta = 0$ deg	Inclination of ground slope behind face of wall, deg (horizontal)
$t = 1.0$	Pavement thickness, ft

Preliminary Wall Dimensioning

Selecting the most optimal wall configuration is an iterative process and depends on site conditions, cost considerations, wall geometry and aesthetics. For this example, the iterative process has been completed and the final wall dimensions are used for design checks.

$H = 20.0$	Design wall height, ft	
$B = 10.0$	Footing base width, ft (2/5H to 3/5H)	
$A = 3.5$	Toe projection, ft (H/8 to H/5)	
$D = 2.0$	Footing thickness, ft (H/8 to H/5)	
	WisDOT policy: H≤10'-0" D_{min}=1'-6" H>10'-0" D_{min}=2'-0"	

Shear Key Dimensioning

$D_{key} = 1.0$	Depth of shear key from bottom of footing, ft	
$D_w = 1.0$	Width of shear key, ft	
$XK = A$	Distance from toe to shear key, ft	

Other Wall Dimensioning

$h' = H - D$	Stem height, ft	$h' = 18.00$
$T_1 = b_1 \dfrac{h'}{12}$	Stem front batter width, ft	$T_1 = 0.375$
$T_2 = b_2 \dfrac{h'}{12}$	Stem back batter width, ft	$T_2 = 0.750$
$T_b = T_1 + T_t + T_2$	Stem thickness at bottom of wall, ft	$T_b = 2.13$
$C = B - A - T_b$	Heel projection, ft	$C = 4.38$
$\theta = \operatorname{atan}\left(\dfrac{12}{b_2}\right)$	Angle of back face of wall to horizontal	$\theta = 87.6\,deg$
$b = 12$	Concrete strip width for design, in	
$y_1 = D_f$	Bottom of footing depth, ft	$y_1 = 4.0$
$y_2 = D_f + D_{key}$	Bottom of shear key depth, ft	$y_2 = 5.0$
$h = H - t + (T_2 + C)\tan(\beta)$	Retained soil height, ft	$h = 19.0$

E14-1.4 Permanent and Transient Loads

In this example, load types DC (dead load components), EV (vertical earth pressure), EH (horizontal earth pressure) and LS (live load surcharge) will be used. Soil above the toe will be ignored as well as its passive resistance. When a shear key is present only the passive soil resistance from the vertical face of the shear key will be included in sliding resistance.

E14-1.4.1 Compute Earth Pressure Coefficients

E14-1.4.1.1 Compute Active Earth Pressure Coefficient

Compute the coefficient of active earth pressure using Coulomb Theory **LRFD [Eq 3.11.5.3-1]**

$$\phi_f = 30.0\,deg$$

$$\beta = 0.0\,deg$$

$$\theta = 87.6\,deg$$

$$\delta = 21.0\,deg$$

$$k_a = \frac{\sin(\theta + \phi_f)^2}{\Gamma\ \sin(\theta)^2\ \sin(\theta - \delta)}$$

$$\Gamma = \left(1 + \sqrt{\frac{\sin(\phi_f + \delta)\ \sin(\phi_f - \beta)}{\sin(\theta - \delta)\ \sin(\theta + \beta)}}\right)^2 \qquad \boxed{\Gamma = 2.726}$$

$$k_a = \frac{\sin(\theta + \phi_f)^2}{\Gamma\ \sin(\theta)^2\ \sin(\theta - \delta)} \qquad \boxed{k_a = 0.314}$$

E14-1.4.1.2 Compute Passive Earth Pressure Coefficient

Compute the coefficient of passive earth pressure using Rankine Theory

$$k_p = \tan\left(45\ deg + \frac{\phi_{fd}}{2}\right)^2 \qquad \boxed{k_p = 3.54}$$

E14-1.4.2 Compute Unfactored Loads

The forces and moments are computed by using Figures E14-1.3-1 and E14-1.3-3 and by their respective load types **LRFD [Tables 3.4.1-1 and 3.4.1-2]**

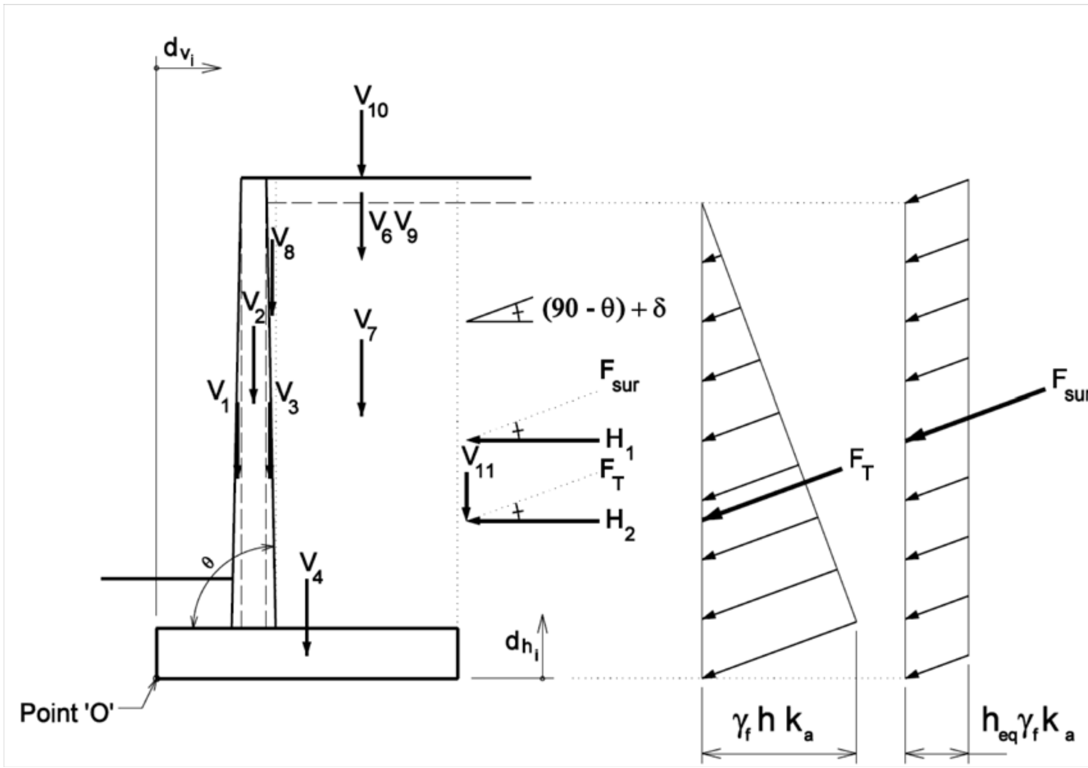


Figure E14-1.4-3
CIP Concrete Wall - External Stability

Active Earth Force Resultant (kip/ft), F_T

$$F_T = \frac{1}{2}\gamma_f h^2 k_a$$ Active earth force resultant (EH) $\boxed{F_T = 6.81}$

Live Load Surcharge Load (kip/ft), F_{sur}

$$F_{sur} = \gamma_f h_{eq} h k_a$$ Live load surcharge resultant (LS) $\boxed{F_{sur} = 1.43}$

Vertical Loads (kip/ft), V_i

$$V_1 = \frac{1}{2} T_1 h' \gamma_c$$ Wall stem front batter (DC) $\boxed{V_1 = 0.51}$

$$V_2 = T_t h' \gamma_c$$ Wall stem (DC) $\boxed{V_2 = 2.70}$

$$V_3 = \frac{1}{2} T_2 h' \gamma_c$$ Wall stem back batter (DC) $\boxed{V_3 = 1.01}$

$V_4 = D\ B\ \gamma_c$ Wall footing (DC) $\boxed{V_4 = 3.00}$

$V_6 = t\left(T_2 + C\right)\gamma_p$ Pavement (DC) $\boxed{V_6 = 0.77}$

$V_7 = C\ (h' - t)\ \gamma_f$ Soil backfill - heel (EV) $\boxed{V_7 = 8.92}$

$V_8 = \frac{1}{2}\ T_2\ (h' - t)\ \gamma_f$ Soil backfill - batter (EV) $\boxed{V_8 = 0.77}$

$$V_9 = \frac{1}{2}\left(T_2 + C\right)\left[\left(T_2 + C\right)\tan(\beta)\right]\gamma_f$$

Soil backfill - backslope (EV) $\boxed{V_9 = 0.00}$

$$V_{10} = h_{eq}\left(T_2 + C\right)\gamma_f$$

Live load surcharge (LS) $\boxed{V_{10} = 1.23}$

$$V_{11} = F_T\ \sin(90\ \text{deg} - \theta + \delta)$$

Active earth force resultant (vertical component - EH) $\boxed{V_{11} = 2.70}$

Moments produced from vertical loads about Point 'O' (kip-ft/ft), MV_i

Moment Arm (ft)		Moment (kip-ft/ft)	
$d_{v1} = A + \frac{2}{3}T_1$	$\boxed{d_{v1} = 3.8}$	$MV_1 = V_1\ d_{v1}$	$\boxed{MV_1 = 1.9}$
$d_{v2} = A + T_1 + \frac{T_t}{2}$	$\boxed{d_{v2} = 4.4}$	$MV_2 = V_2\ d_{v2}$	$\boxed{MV_2 = 11.8}$
$d_{v3} = A + T_1 + T_t + \frac{T_2}{3}$	$\boxed{d_{v3} = 5.1}$	$MV_3 = V_3\ d_{v3}$	$\boxed{MV_3 = 5.2}$
$d_{v4} = \frac{B}{2}$	$\boxed{d_{v4} = 5.0}$	$MV_4 = V_4\ d_{v4}$	$\boxed{MV_4 = 15.0}$
$d_{v6} = B - \left(\frac{T_2 + C}{2}\right)$	$\boxed{d_{v6} = 7.4}$	$MV_6 = V_6\ d_{v6}$	$\boxed{MV_6 = 5.7}$
$d_{v7} = B - \frac{C}{2}$	$\boxed{d_{v7} = 7.8}$	$MV_7 = V_7\ d_{v7}$	$\boxed{MV_7 = 69.7}$

$$d_{v8} = A + T_1 + T_t + \frac{2T_2}{3} \qquad \boxed{d_{v8} = 5.4} \qquad MV_8 = V_8\, d_{v8} \qquad \boxed{MV_8 = 4.1}$$

$$d_{v9} = A + T_1 + T_t + \frac{2\left(T_2 + C\right)}{3} \qquad \boxed{d_{v9} = 8.3} \qquad MV_9 = V_9\, d_{v9} \qquad \boxed{MV_9 = 0.0}$$

$$d_{v10} = B - \left(\frac{T_2 + C}{2}\right) \qquad \boxed{d_{v10} = 7.4} \qquad MV_{10} = V_{10}\, d_{v10} \qquad \boxed{MV_{10} = 9.1}$$

$$d_{v11} = B \qquad \boxed{d_{v11} = 10.0} \qquad MV_{11} = V_{11}\, d_{v11} \qquad \boxed{MV_{11} = 27.0}$$

Horizontal Loads (kip/ft), H_i

$$H_1 = F_{sur} \cos\left(90\ deg - \theta + \delta\right)$$

Live load surcharge (LS) $\boxed{H_1 = 1.32}$

$$H_2 = F_T \cos\left(90\ deg - \theta + \delta\right)$$

Active earth force
(horizontal component) (EH) $\boxed{H_2 = 6.25}$

Moments produced from horizontal loads about about Point 'O' (kip-ft/ft), MH_i

Moment Arm (ft) Moment (kip-ft/ft)

$$d_{h1} = \frac{h}{2} \qquad \boxed{d_{h1} = 9.5} \qquad MH_1 = H_1\, d_{h1} \qquad \boxed{MH_1 = 12.5}$$

$$d_{h2} = \frac{h}{3} \qquad \boxed{d_{h2} = 6.3} \qquad MH_2 = H_2\, d_{h2} \qquad \boxed{MH_2 = 39.6}$$

Summary of Unfactored Forces & Moments:

Load			Moment Arm		Moment		LRFD Load Type
Item	Description	Value (kip/ft)	Item	Value (ft)	Item	Value (kip-ft/ft)	
V_1	Wall stem front batter	0.51	d_{v1}	3.8	MV_1	1.9	DC
V_2	Wall stem	2.70	d_{v2}	4.4	MV_2	11.8	DC
V_3	Wall stem back batter	1.01	d_{v3}	5.1	MV_3	5.2	DC
V_4	Wall footing	3.00	d_{v4}	5.0	MV_4	15.0	DC
V_6	Pavement	0.77	d_{v6}	7.4	MV_6	5.7	DC
V_7	Soil backfill	8.92	d_{v7}	7.8	MV_7	69.7	EV
V_8	Soil backfill	0.77	d_{v8}	5.4	MV_8	4.1	EV
V_9	Soil backfill	0.00	d_{v9}	8.3	MV_9	0.0	EV
V_{10}	Live load surcharge	1.23	d_{v10}	7.4	MV_{10}	9.2	LS
V_{11}	Active earth pressure	2.70	d_{v11}	10.0	MV_{11}	27.0	EH

Table E14-1.4-1
Unfactored Vertical Forces & Moments

Load			Moment Arm		Moment		LRFD Load Type
Item	Description	Value (kip/ft)	Item	Value (ft)	Item	Value (kip-ft/ft)	
H_1	Live load surcharge	1.32	d_{h1}	9.5	MH_1	12.5	LS
H_2	Active earth force	6.25	d_{h2}	6.3	MH_2	39.6	EH

Table E14-1.4-2
Unfactored Horizontal Forces & Moments

E14-1.4.3 Summarize Applicable Load and Resistance Factors

Maximum and minimum load factors shall be used to determine the extreme load effects. WisDOT's policy is to set all the load modifiers to zero($n = 1.0$). Factored loads and moments for each limit state are calculated by applying the appropriate load factors **LRFD [Tables 3.4.1-1 and 3.4.1-2]**. The following load combinations will be used in this example:

Load Combination	γ_{DC}	γ_{EV}	γ_{LS_V}	γ_{LS_H}	γ_{EH}	Application
Strength Ia	0.90	1.00	-	1.75	1.50	Sliding, Eccentricity
Strength Ib	1.25	1.35	1.75	1.75	1.50	Bearing, Wall Strength
Service I	1.00	1.00	1.00	1.00	1.00	Wall Crack Control

Table E14-1.4-3
Load Combinations

Load Combination Assumptions:

- Live load surcharge stabilizing loads (if applicable) are ignored for overturning and sliding analyses. Live load surcharge is used to compute maximum bearing pressure, wall strength and overall (global) stability.
- Minimum horizontal earth pressure ,$\gamma_{EH(min)}$ = 0.9, will not control in this example based on B/H and lateral load inclination, but should be checked.
- Vertical loads from vehicle collision need not be applied with transverse loads. By inspection, transverse loads will control Extreme Event Load Combination for this example.
- Component load factors shall remain consistent throughout calculations. For example, the active earth force resultant (F_T) can be broken into component forces of either $V_{10}\gamma_{EH(max)}$ and $H_2\gamma_{EH(max)}$ or $V_{10}\gamma_{EH(min)}$ and $H_2\gamma_{EH(min)}$, not $V_{10}\gamma_{EH(min)}$ and $H_2\gamma_{EH(max)}$.

The loads discussed and tabulated previously can now be factored by the appropriate load factors and combined to determine the governing limit states for each design check.

E14-1.4.4 Compute Factored Loads and Moments

Unfactored loads by load type (kip/ft)

$V_{DC} = V_1 + V_2 + V_3 + V_4 + V_6$ $\boxed{V_{DC} = 8.0}$

$V_{EV} = V_7 + V_8 + V_9$ $\boxed{V_{EV} = 9.7}$

$V_{LS} = V_{10}$ $\boxed{V_{LS} = 1.2}$

$V_{EH} = V_{11}$ $\boxed{V_{EH} = 2.7}$

$H_{LS} = H_1$ $\boxed{H_{LS} = 1.3}$

$H_{EH} = H_2$ $\boxed{H_{EH} = 6.3}$

Unfactored moments by load type (kip-ft/ft)

$M_{DC} = MV_1 + MV_2 + MV_3 + MV_4 + MV_6$ $\boxed{M_{DC} = 39.6}$

$M_{EV} = MV_7 + MV_8 + MV_9$ $\boxed{M_{EV} = 73.8}$

$M_{LS1} = MV_{10}$ $\boxed{M_{LS1} = 9.1}$

$M_{EH1} = MV_{11}$ $\boxed{M_{EH1} = 27.0}$

$M_{LS2} = MH_1$ $\boxed{M_{LS2} = 12.5}$

$M_{EH2} = MH_2$ $\boxed{M_{EH2} = 39.6}$

Factored vertical loads by limit state (kip/ft)

$V\_Ia = n\left(0.90V_{DC} + 1.00V_{EV} + 0.00\ V_{LS} + 1.50\ V_{EH}\right)$ $\boxed{V\_Ia = 20.9}$

$V\_Ib = n\left(1.25V_{DC} + 1.35V_{EV} + 1.75\ V_{LS} + 1.50\ V_{EH}\right)$ $\boxed{V\_Ib = 29.3}$

$V\_Ser = n\left(1.00V_{DC} + 1.00V_{EV} + 1.00\ V_{LS} + 1.00\ V_{EH}\right)$ $\boxed{V\_Ser = 21.6}$

Factored horizontal loads by limit state (kip/ft)

$$H\_Ia = n\left(1.75H_{LS} + 1.50H_{EH}\right)$$ $\boxed{H\_Ia = 11.7}$

$$H\_Ib = n\left(1.75H_{LS} + 1.50H_{EH}\right)$$ $\boxed{H\_Ib = 11.7}$

$$H\_Ser = n\left(1.00H_{LS} + 1.00H_{EH}\right)$$ $\boxed{H\_Ser = 7.6}$

Factored moments produced by vertical Loads by limit state (kip-ft/ft)

$$MV\_Ia = n\left(0.90M_{DC} + 1.00M_{EV} + 0.00M_{LS1} + 1.50\ M_{EH1}\right)$$ $\boxed{MV\_Ia = 150.0}$

$$MV\_Ib = n\left(1.25M_{DC} + 1.35M_{EV} + 1.75M_{LS1} + 1.50\ M_{EH1}\right)$$ $\boxed{MV\_Ib = 205.8}$

$$MV\_Ser = n\left(1.00M_{DC} + 1.00M_{EV} + 1.00M_{LS1} + 1.00\ M_{EH1}\right)$$ $\boxed{MV\_Ser = 149.6}$

Factored moments produced by horizontal loads by limit state (kip-ft/ft)

$$MH\_Ia = n\left(1.75M_{LS2} + 1.50\ M_{EH2}\right)$$ $\boxed{MH\_Ia = 81.3}$

$$MH\_Ib = n\left(1.75M_{LS2} + 1.50\ M_{EH2}\right)$$ $\boxed{MH\_Ib = 81.3}$

$$MH\_Ser = n\left(1.00M_{LS2} + 1.00\ M_{EH2}\right)$$ $\boxed{MH\_Ser = 52.1}$

Load Combination	Vert. Loads V (kips/ft)	Moments MV (kip-kip/ft)	Horiz. Loads H (kips/ft)	Moments MH (kip-kip/ft)
Strength Ia	20.9	150.0	11.7	81.3
Strength Ib	29.3	205.8	11.7	81.3
Service I	21.6	149.6	7.6	52.1

Table E14-1.4-4
Summary of Factored Loads & Moments

E14-1.5 Compute Bearing Resistance, q_R

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

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

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

$\Sigma M_R = MV\_Ser$ $\boxed{\Sigma M_R = 149.6}$ Summation of resisting moments for Service I

$\Sigma M_O = MH\_Ser$ $\boxed{\Sigma M_O = 52.1}$ Summation of overturning moments for Service I

$\Sigma V = V\_Ser$ $\boxed{\Sigma V = 21.6}$ Summation of vertical loads for Service I

$x = \dfrac{\Sigma M_R - \Sigma M_O}{\Sigma V}$ Distance from Point "O" the resultant intersects the base $\boxed{x = 4.51}$ ft

Compute the wall eccentricity

$e = \dfrac{B}{2} - x$ $\boxed{e = 0.49}$ ft

Define the foundation layout

$B' = B - 2\,e$ Footing width $\boxed{B' = 9.0}$ ft

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

$H' = H\_Ser$ Summation of horizontal loads for Service I $\boxed{H' = 7.6}$ kip/ft

$V' = V\_Ser$ Summation of vertical loads for Service I $\boxed{V' = 21.6}$ kip/ft

$D_f = 4.00$ Footing embedment

$\theta' = \mathrm{atan}\left(\dfrac{V'}{H'}\right)$ Direction of resultant from horizontal $\boxed{\theta' = 70.7\,\mathrm{deg}}$

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

$\phi_{fd} = 34.0\,\mathrm{deg}$ $N_q = 29.4$ $N_c = 42.16$ $N_\gamma = 41.06$

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

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

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

$s_q = 1 + \left(\dfrac{B'}{L'}\tan(\phi_{fd})\right)$ $\boxed{s_q = 1.07}$

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

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

$$n = \frac{2 + \frac{L'}{B'}}{1 + \frac{L'}{B'}} \cos(\theta')^2 + \frac{2 + \frac{B'}{L'}}{1 + \frac{B'}{L'}} \sin(\theta')^2$$ $\boxed{n = 1.82}$

$$i_q = \left(1 - \frac{H'}{V' + c_{fd}\, B'\, L' \frac{1}{\tan(\phi_{fd})}}\right)^n$$ $\boxed{i_q = 0.46}$

$$i_\gamma = \left(1 - \frac{H'}{V' + c_{fd}\, B'\, L' \frac{1}{\tan(\phi_{fd})}}\right)^{n+1}$$ $\boxed{i_\gamma = 0.30}$

$$i_c = i_q - \left(\frac{1 - i_q}{N_q - 1}\right)$$ For $\phi_{fd} > 0$: $\boxed{i_c = 0.44}$

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

$d_q = 1.00$

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

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

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

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

$N_{cm} = N_c\, s_c\, i_c$ $\boxed{N_{cm} = 19.7}$

$N_{qm} = N_q\, s_q\, d_q\, i_q$ $\boxed{N_{qm} = 14.3}$

$N_{\gamma m} = N_\gamma\, s_\gamma\, i_\gamma$ $\boxed{N_{\gamma m} = 11.7}$

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

$q_n = c_{fd}\, N_{cm} + \gamma_{fd}\, D_f\, N_{qm}\, C_{wq} + 0.5\, \gamma_{fd}\, B'\, N_{\gamma m}\, C_{w\gamma}$ $\boxed{q_n = 13.21}$ ksf/ft

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

$\phi_b = 0.45$

$q_R = \phi_b\, q_n$ $\boxed{q_R = 5.95}$ ksf/ft

E14-1.6 Evaluate External Stability of Wall

Three potential external failure mechanisms will be considered in this example. These failures include bearing, limiting eccentricity and sliding. Global (overall) stability requirements are assumed to have been satisfied in prior calculations. Design calculations will be carried out for the governing limit states only.

E14-1.6.1 Bearing Resistance at Base of the Wall

The following calculations are based on **Strength Ib**:

Compute resultant location (distance from Point 'O' Figure E14-1.4-3)

$\Sigma M_R = MV\_Ib$ Summation of resisting moments for Strength Ib

$\Sigma M_O = MH\_Ib$ Summation of overturning moments for Strength Ib

$\Sigma V = V\_Ib$ Summation of vertical loads for Strength Ib

$\Sigma M_R = 205.8$ kip-ft/ft

$\Sigma M_O = 81.3$ kip-ft/ft

$\Sigma V = 29.3$ kip/ft

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

$\boxed{x = 4.25}$ ft

Compute the wall eccentricity

$$e = \frac{B}{2} - x$$

$\boxed{e = 0.75}$ ft

Note: The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e. When the foundation eccentricity is negative the actual bearing width, B, will be used.

Compute the ultimate bearing stress

$$\sigma_V = \frac{\Sigma V}{B - 2\,e}$$

$\boxed{\sigma_V = 3.44}$ ksf/ft

Factored bearing resistance

$\boxed{q_R = 5.95}$ ksf/ft

<u>Capacity:Demand Ratio (CDR)</u>

$$CDR_{Bearing1} = \frac{q_R}{\sigma_V}$$

$\boxed{CDR_{Bearing1} = 1.73}$

Is the CDR ≥ 1.0 ?

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

E14-1.6.2 Limiting Eccentricity at Base of the Wall

The location of the resultant of the reaction forces is limited to the middle one-half of the base width for a soil foundation (i.e., e_{max} = L/4) **LRFD [11.6.3.3]**. The following calculations are based on **Strength Ia**:

Maximum eccentricity

$$e_{max} = \frac{B}{4}$$ $\boxed{e_{max} = 2.50}$ ft

Compute resultant location (distance from Point 'O' Figure E14-1.4.3)

$\Sigma M_R = MV\_Ia$ Summation of resisting moments for Strength Ia

$\Sigma M_O = MH\_Ia$ Summation of overturning moments for Strength Ia

$\Sigma V = V\_Ia$ Summation of vertical loads for Strength Ia

$\Sigma M_R = 150.0$ kip-ft/ft

$\Sigma M_O = 81.3$ kip-ft/ft

$\Sigma V = 20.9$ kip/ft

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

$\boxed{x = 3.29}$ ft

Compute the wall eccentricity

$$e = \frac{B}{2} - x$$ $\boxed{e = 1.71}$ ft

<u>Capacity:Demand Ratio (CDR)</u>

$$CDR_{Eccentricity1} = \frac{e_{max}}{e}$$ $\boxed{CDR_{Eccentricity1} = 1.46}$

Is the $CDR \geq 1.0$? $\boxed{check = \text{"OK"}}$

E14-1.6.3 Sliding Resistance at Base of the Wall

For sliding failure, the horizontal force effects, R_u, is checked against the sliding resistance, R_R, where $R_R=\phi R_n$ **LRFD [10.6.3.4]**. If sliding resistance is not adequate a shear key will be investigated. The following calculations are based on **Strength Ia**:

Factored Sliding Force, R_u

$R_u = H\_Ia$ $\boxed{R_u = 11.7}$ kip/ft

Sliding Resistance, R_R

$$R_R = \phi_s R_n = \phi_\tau R_\tau + \phi_{pe} R_{ep}$$

Compute sliding resistance between soil and foundation, $\phi_\tau\ R_\tau$

$\Sigma V = V\_Ia$ $\boxed{\Sigma V = 20.9}$ kip/ft

$\phi_\tau = 1.00$ $\phi_{\tau 1} = 0.80$ $\phi_{\tau 2} = 1.00$

$R_\tau = \Sigma V \tan(\phi_{fd}) \left(\phi_{\tau 1} \dfrac{D_w}{B} + \phi_{\tau 2} \dfrac{B - D_w}{B}\right)$ $\boxed{R_\tau = 13.8}$ kip/ft

$\boxed{\phi_\tau\ R_\tau = 13.8}$ kip/ft

Compute passive resistance throughout the design life of the wall, $\phi_{ep}\ R_{ep}$

$r_{ep1} = k_p\ \gamma_{fd}\ y_1$ Nominal passive pressure at y_1 $\boxed{r_{ep1} = 1.70}$ kip/ft

$r_{ep2} = k_p\ \gamma_{fd}\ y_2$ Nominal passive pressure at y_2 $\boxed{r_{ep2} = 2.12}$ kip/ft

$R_{ep} = \dfrac{r_{ep1} + r_{ep2}}{2}(y_2 - y_1)$ $\boxed{R_{ep} = 1.9}$ kip/ft

$\phi_{ep} = 0.50$ $\boxed{\phi_{ep}\ R_{ep} = 1.0}$ kip/ft

Compute nominal resistance against failure by sliding, R_n

$R_n = \phi_\tau\ R_\tau + \phi_{ep}\ R_{ep}$ $\boxed{R_n = 14.8}$ kip/ft

Compute factored resistance against failure by sliding, R_R

$\phi = 1.00$

$R_R = \phi\ R_n$ $\boxed{R_R = 14.8}$ kip/ft

<u>Capacity:Demand Ratio (CDR)</u>

$CDR_{Sliding1} = \dfrac{R_R}{R_u}$ $\boxed{CDR_{Sliding1} = 1.27}$

Is the CDR $\geq$ 1.0 ? $\boxed{\text{check = "OK"}}$

E14-1.7 Evaluate Wall Structural Design

Note: CIP concrete walls are a non-proprietary wall system and the structural design computations shall be performed by the wall designer.

Wall structural design computations for shear and flexure will be considered in this example. The critical sections for flexure are taken at the front, back and bottom of them stem. For simplicity, critical sections for shear will be taken at the critical sections used for flexsure. In actuality, the toe and stem may be designed for shear at the effective depth away from the face. Crack control and temperature and shrinkage considerations will also be included.

E14-1.7.1 Evaluate Heel Strength

E14-1.7.1.1 Evaluate Heel Shear Strength

For **Strength Ib**:

$$V_u = 1.25\left(\frac{C}{B}V_4 + V_6\right) + 1.35\left(V_7 + V_8 + V_9\right) + 1.75\left(V_{10}\right) + 1.50\left(V_{11}\right)$$

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

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.8.3.3]**

$V_{n1} = V_c$ **LRFD [Eq 5.8.3.3-1]**

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

$V_{n2} = 0.25\ f'_c\ b_v\ d_v$ **LRFD [Eq 5.8.3.3-2]**

Compute the shear resistance due to concrete, V_c :

$cover = 2.0$ in

$s = 7.0$ in (bar spacing)

$Bar_{No} = 6$ (transverse bar size)

$Bar_D = 0.750$ in (transverse bar diameter)

$Bar_A = 0.440$ in^2 (transverse bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}}$$

$\boxed{A_s = 0.75}$ in^2/ft

$$d_s = D\ 12 - cover - \frac{Bar_D}{2}$$

$\boxed{d_s = 21.6}$ in

$$a = \frac{A_s\ f_y}{0.85\ f'_c\ b}$$

$\boxed{a = 1.3}$ in

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

$$\phi_F = \begin{cases} 0.75 & \text{if } \frac{d_s}{c} < \frac{5}{3} \\ 0.65 + 0.15\left(\frac{d_s}{c} - 1\right) & \text{if } \frac{5}{3} \le \frac{d_s}{c} \le \frac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$ $\boxed{\phi_F = 0.90}$

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

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

$$f_r = 0.37\sqrt{f'_c}$$ $\boxed{f_r = 0.692}$ ksi

$$I_g = \frac{1}{12}\, b\, (D\, 12)^3$$ $\boxed{I_g = 13824}$ in^4

$$y_t = \frac{1}{2}\, D\, 12$$ $\boxed{y_t = 12.00}$ in

$$S_c = \frac{I_g}{y_t}$$ $\boxed{S_c = 1152}$ in^3

$$M_{cr} = S_c\, f_r\, \frac{1}{12}$$ $\boxed{M_{cr} = 66.5}$ kip-ft/ft

$\boxed{1.2\, M_{cr} = 79.7}$ kip-ft/ft

$\boxed{1.33\, M_u = 63.7}$ kip-ft/ft

Is M_r greater than the lesser value of $1.2*M_{cr}$ and $1.33*M_u$? $\boxed{check = \text{"OK"}}$

E14-1.7.2 Evaluate Toe Strength

The structural design of the footing toe is calculated using a linear contact stress distribution for bearing for all soil and rock conditions.

E14-1.7.2.1 Evaluate Toe Shear Strength

For **Strength Ib**:

$\Sigma M_R = MV\_Ib$ $\boxed{\Sigma M_R = 205.8}$ kip-ft/ft

$\Sigma M_O = MH\_Ib$ $\boxed{\Sigma M_O = 81.3}$ kip-ft/ft

$\Sigma V = V\_Ib$ $\boxed{\Sigma V = 29.3}$ kip/ft

$$x = \frac{\Sigma M_R - \Sigma M_O}{\Sigma V}$$ $\boxed{x = 4.3}$ ft

$$e = \max\left(0, \frac{B}{2} - x\right)$$ $\boxed{e = 0.75}$ ft

$$\sigma_{max} = \frac{\Sigma V}{B}\left(1 + 6\frac{e}{B}\right)$$ $\boxed{\sigma_{max} = 4.24}$ ksf/ft

$$\sigma_{min} = \frac{\Sigma V}{B}\left(1 - 6\frac{e}{B}\right)$$ $\boxed{\sigma_{min} = 1.62}$ ksf/ft

Calculate the average stress on the toe

$$\sigma_v = \frac{\sigma_{max} + \left[\sigma_{min} + \frac{B - A}{B}\left(\sigma_{max} - \sigma_{min}\right)\right]}{2}$$ $\boxed{\sigma_v = 3.78}$ ksf/ft

$V_u = \sigma_v A$ $\boxed{V_u = 13.2}$ kip/ft

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.8.3.3]**

$V_{n1} = V_c$ **LRFD [Eq 5.8.3.3-1]**

in which: $V_c = 0.0316\,\beta\sqrt{f'_c}\,b_v\,d_v$

$V_{n2} = 0.25\,f'_c\,b_v\,d_v$ **LRFD [Eq 5.8.3.3-2]**

Design footing toe for shear

$cover = 3.0$ in

$s = 9.0$ in (bar spacing)

$Bar_{No} = 5$ (transverse bar size)

$Bar_D = 0.63$ in (transverse bar diameter)

$Bar_A = 0.31$ in² (transverse bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}}$$ $\boxed{A_s = 0.41}$ in²/ft

$$d_s = D\,12 - cover - \frac{Bar_D}{2}$$ $\boxed{d_s = 20.7}$ in

$$a = \frac{A_s\,f_y}{0.85\,f'_c\,b}$$ $\boxed{a = 0.7}$ in

$$d_{v1} = d_s - \frac{a}{2}$$ $\boxed{d_{v1} = 20.3}$ in

$$d_{v2} = 0.9\,d_s$$ $\boxed{d_{v2} = 18.6}$ in

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

$$d_v = \max(d_{v1}, d_{v2}, d_{v3})$$ $\boxed{d_v = 20.3}$ in

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

$$\beta = 2.0$$

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

$$V_{n1} = V_c$$ $\boxed{V_{n1} = 28.9}$ kip/ft

$$V_{n2} = 0.25\,f'_c\,b\,d_v$$ $\boxed{V_{n2} = 213.6}$ kip/ft

$$V_n = \min(V_{n1}, V_{n2})$$ $\boxed{V_n = 28.9}$ kip/ft

$$V_r = \phi_v\,V_n$$ $\boxed{V_r = 26.0}$ kip/ft

$\boxed{V_u = 13.2}$ kip/ft

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

E14-1.7.1.2 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

$$\phi_F = \begin{vmatrix} 0.75 \text{ if } \frac{d_s}{c} < \frac{5}{3} \\ 0.65 + 0.15\left(\frac{d_s}{c} - 1\right) \text{ if } \frac{5}{3} \le \frac{d_s}{c} \le \frac{8}{3} \\ 0.90 \text{ otherwise} \end{vmatrix}$$ $\boxed{\phi_F = 0.90}$

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]**:

$$f_r = 0.37 \sqrt{f'_c}$$ $\boxed{f_r = 0.692}$ ksi

$$I_g = \frac{1}{12} b (D\ 12)^3$$ $\boxed{I_g = 13824}$ in^4

$$y_t = \frac{1}{2} D\ 12$$ $\boxed{y_t = 12.00}$ in

$$S_c = \frac{I_g}{y_t}$$ $\boxed{S_c = 1152}$ in^3

$$M_{cr} = S_c f_r \frac{1}{12}$$ $\boxed{M_{cr} = 66.5}$ kip-ft/ft

$\boxed{1.2\ M_{cr} = 79.7}$ kip-ft/ft

$\boxed{1.33\ M_u = 30.8}$ kip-ft/ft

Is M_r greater than the lesser value of $1.2*M_{cr}$ and $1.33*M_u$?

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

E14-1.7.3 Evaluate Stem Strength

Unfactored Stem Horizontal Loads and Moments:

$$H_1 = \gamma_f\, h_{eq}\, (h' - t)\, k_a \cos(90\ \deg - \theta + \delta)$$ $\boxed{H_1 = 1.2}$ kip/ft

$$H_2 = \frac{1}{2}\gamma_f\,(h' - t)^2\, k_a \cos(90\ \deg - \theta + \delta)$$ $\boxed{H_2 = 5.0}$ kip/ft

$$M_1 = H_1\left(\frac{h' - t}{2}\right)$$ $\boxed{M_1 = 10.0}$ kip-ft/ft

$$M_2 = H_2\left(\frac{h' - t}{3}\right)$$ $\boxed{M_2 = 28.4}$ kip-ft/ft

Factored Stem Horizontal Loads and Moments:

for **Strength Ib**:

$$H_{u1} = 1.75\,H_1 + 1.50\,H_2$$ $\boxed{H_{u1} = 9.6}$ kip/ft

$$M_{u1} = 1.75\,M_1 + 1.50\,M_2$$ $\boxed{M_{u1} = 60.0}$ kip-ft/ft

for **Service I**:

$$H_{u3} = 1.00\,H_1 + 1.00\,H_2$$ $\boxed{H_{u3} = 6.2}$ kip/ft

$$M_{u3} = 1.00\,M_1 + 1.00\,M_2$$ $\boxed{M_{u3} = 38.4}$ kip-ft/ft

E14-1.7.3.1 Evaluate Stem Shear Strength at Footing

$$V_u = H_{u1}$$ $\boxed{V_u = 9.6}$ kip/ft

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.8.3.3]**

$V_{n1} = V_c$ **LRFD [Eq 5.8.3.3-1]**

where: $V_c = 0.0316\,\beta\,\sqrt{f'_c}\,b_v\,d_v$

$V_{n2} = 0.25\,f'_c\,b_v\,d_v$ **LRFD [Eq 5.8.3.3-2]**

Compute the shear resistance due to concrete, V_c :

$cover = 2.0$ in

$s = 10.0$ in (bar spacing)

$Bar_{No} = 8$ (transverse bar size)

$Bar_D = 1.00$ in (transverse bar diameter)

$Bar_A = 0.79$ in^2 (transverse bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}} \qquad \boxed{A_s = 0.95} \ in^2/ft$$

$$d_s = T_b\,12 - cover - \frac{Bar_D}{2} \qquad \boxed{d_s = 23.0} \ in$$

$$a = \frac{A_s\,f_y}{0.85\,f'_c\,b} \qquad \boxed{a = 1.6} \ in$$

$$d_{v1} = d_s - \frac{a}{2} \qquad \boxed{d_{v1} = 22.2} \ in$$

$$d_{v2} = 0.9\,d_s \qquad \boxed{d_{v2} = 20.7} \ in$$

$$d_{v3} = 0.72\,T_b\,12 \qquad \boxed{d_{v3} = 18.4} \ in$$

$$d_v = \max(d_{v1}, d_{v2}, d_{v3}) \qquad \boxed{d_v = 22.2} \ in$$

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

E14-1.7.3.2 Evaluate Stem Flexural Strength at Footing

$M_u = M_{u1}$ $\boxed{M_u = 60.0}$ kip-ft/ft

Calculate the capacity of the stem in flexure at the face of the footing:

$M_n = A_s\, f_y \left(d_s - \frac{a}{2}\right) \frac{1}{12}$ $\boxed{M_n = 105.2}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$\beta_1 = 0.85$

$c = \frac{a}{\beta_1}$ $\boxed{c = 1.87}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \frac{d_s}{c} < \frac{5}{3} \\ 0.65 + 0.15\left(\frac{d_s}{c} - 1\right) & \text{if } \frac{5}{3} \le \frac{d_s}{c} \le \frac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$ $\boxed{\phi_F = 0.90}$

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

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

$f_r = 0.37\sqrt{f'_c}$ $\boxed{f_r = 0.69}$ ksi

$I_g = \frac{1}{12}\, b\, (T_b\, 12)^3$ $\boxed{I_g = 16581}$ 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

$M_{cr\_s} = S_c\, f_r\, \frac{1}{12}$ $\boxed{M_{cr\_s} = 75.0}$ kip-ft/ft

$\boxed{1.2\ M_{cr\_s} = 90.0}$ kip-ft/ft

$\boxed{1.33\ M_u = 79.9}$ kip-ft/ft

Is M_r greater than the lesser value of $1.2*M_{cr}$ and $1.33*M_u$?

$\boxed{check = "OK"}$

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

$$\rho = \frac{A_s}{d_s\ b}$$ $\boxed{\rho = 0.00343}$

$$n = \frac{E_s}{E_c}$$ $\boxed{n = 8.09}$

$$k = \sqrt{(\rho\ n)^2 + 2\ \rho\ n} - \rho\ n$$ $\boxed{k = 0.210}$

$$j = 1 - \frac{k}{3}$$ $\boxed{j = 0.930}$

$$d_c = cover + \frac{Bar_D}{2}$$ $\boxed{d_c = 2.5}$ in

$$f_{ss} = \frac{M_{u3}}{A_s\ j\ d_s}\ 12$$ $\boxed{f_{ss} = 22.7}$ ksi

$$h = T_b\ 12$$

$$\beta_s = 1 + \frac{d_c}{0.7\left(h - d_c\right)}$$ $\boxed{\beta_s = 1.2}$

$\gamma_e = 1.0$ 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}? $\boxed{check = "OK"}$

E14-1.7.3.3 Transfer of Force at Base of Stem

Specification requires that the transfer of lateral forces from the stem to the footing be in accordance with the shear-transfer provisions of **LRFD [5.8.4]**. That calculation will not be presented. Refer to E13-1.9.3 for a similar computation.

E14-1.7.4 Temperature and Shrinkage Steel

E14-1.7.4.1 Temperature and Shrinkage Steel for Footing

The footing will not be exposed to daily temperature changes. Thus temperature and shrinkage steel is not required. However, #4 bars at 18" o.c. (max) are placed longitudinally to serve as spacers.

E14-1.7.4.2 Temperature and Shrinkage Steel of Stem

The stem will be exposed to daily temperature changes. In accordance with **LRFD [5.10.8]** the stem shall provide temperature and shrinkage steel on each face and in each direction as calculated below:

$s = 18.0$ in (bar spacing)

$Bar_{No} = 4$ (bar size)

$Bar_D = 0.50$ in (temperature and shrinkage bar diameter)

$Bar_A = 0.20$ in^2 (temperature and shrinkage bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}}$$ (temperature and shrinkage provided)

$\boxed{A_s = 0.13}$ in^2/ft

$b_s = (H - D)\ 12$ least width of stem $\boxed{b_s = 216.0}$ in

$h_s = T_t\ 12$ least thickness of stem $\boxed{h_s = 12.0}$ in

$$A_{ts} = \frac{1.3\ b_s\ h_s}{2\left(b_s + h_s\right)\ f_y}$$ Area of reinforcement per foot, on each face and in each direction $\boxed{A_{ts} = 0.12}$ in^2/ft

Is $0.11 \leq A_s \leq 0.60$? $\boxed{check = \text{"OK"}}$

Is $A_s > A_{ts}$? $\boxed{check = \text{"OK"}}$

Check the maximum spacing requirements

$s_1 = \min(3\ h_s, 18)$ $\boxed{s_1 = 18.0}$ in

$s_2 = \begin{cases} 12 & \text{if } h_s > 18 \\ s_1 & \text{otherwise} \end{cases}$ For walls and footings (in) $\boxed{s_2 = 18.0}$ in

$s_{max} = \min(s_1, s_2)$ $\boxed{s_{max} = 18.0}$ in

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

E14-1.8 Summary of Results

E14-1.8.1 Summary of External Stability

Based on the defined project parameters the following external stability checks have been satisfied:

External Check	CDR
	Strength
Sliding	1.27
Eccentricity	1.45
Bearing	1.73

Table E14-1.8-1
Summary of External Stability Computations

E14-1.8.2 Summary of Wall Strength Design

The required wall reinforcing from the previous computations are presented in Figure E14-1.9-1.

E14-1.8.3 Drainage Design

Drainage requirements shall be investigated and detailed accordingly. In this example drainage requirements are met by providing granular, free draining backfill materail with a pipe underdrain located at the bottom of the wall (Assumed wall is adjacent to sidewalk) as shown in Figure E14-1.9-1.

E14-1.9 Final CIP Concrete Wall Schematic

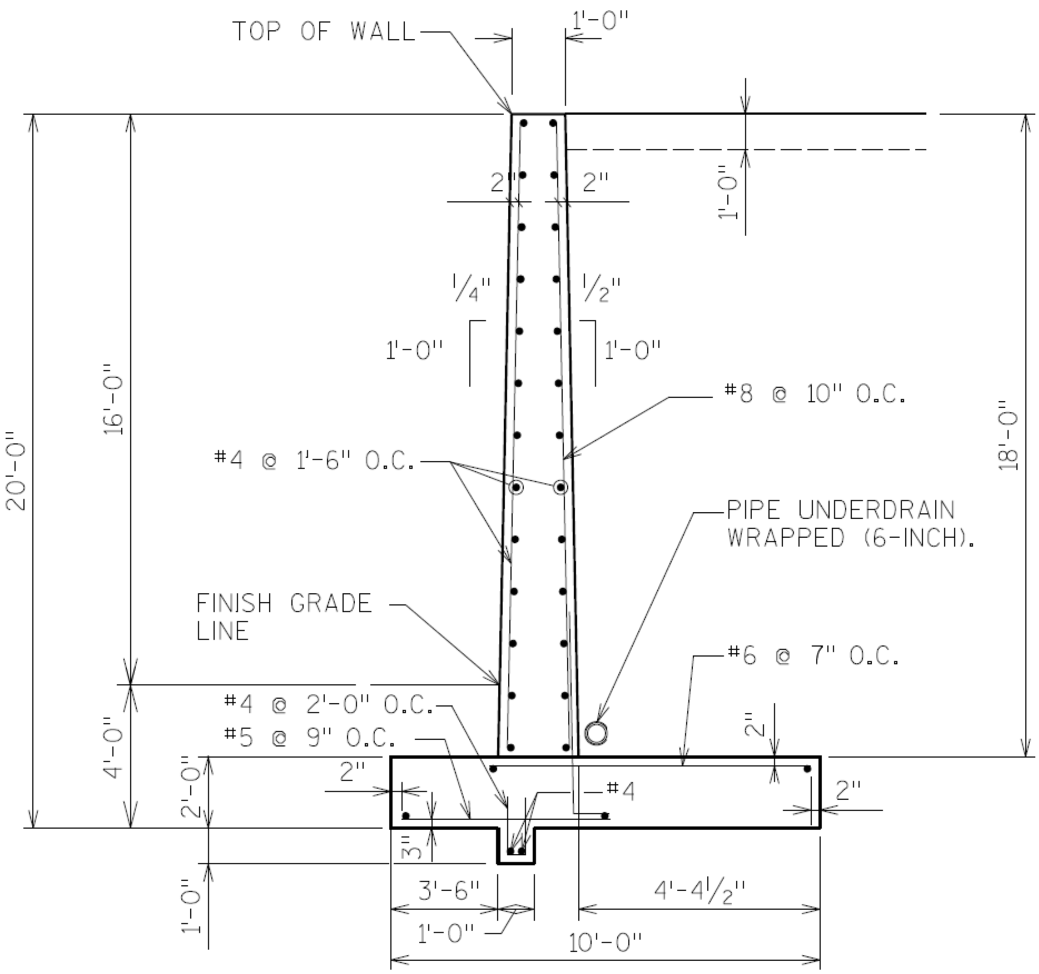


Figure E14-1.9-1
Cast-In-Place Wall Schematic

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E14-2 Precast Panel Steel Reinforced MSE Wall, LRFD

General

This example shows design calculations for MSE wall with precast concrete panel facings conforming to the LRFD Bridge Design Specifications and the WisDOT Bridge Manual. *(Example is current through LRFD Fifth Edition - 2010)*

Sample design calculations for external stability (sliding, eccentricity and bearing) and internal stability (soil reinforcement stress and pullout) will be presented. The overall stability and settlement calculations will not be shown in this example, but are required.

Design steps presented in 14.6.3.3 are used for the wall design.

E14-2.1 Establish Project Requirements

The following MSE wall shall have compacted freely draining soil in the reinforced zone and will be reinforced with metallic (inextensible) strips as shown in Figure E14-2.1-1. External stability is the designer's (WisDOT/Consultant) responsibility and internal stability and structural components are the contractors responsibility.

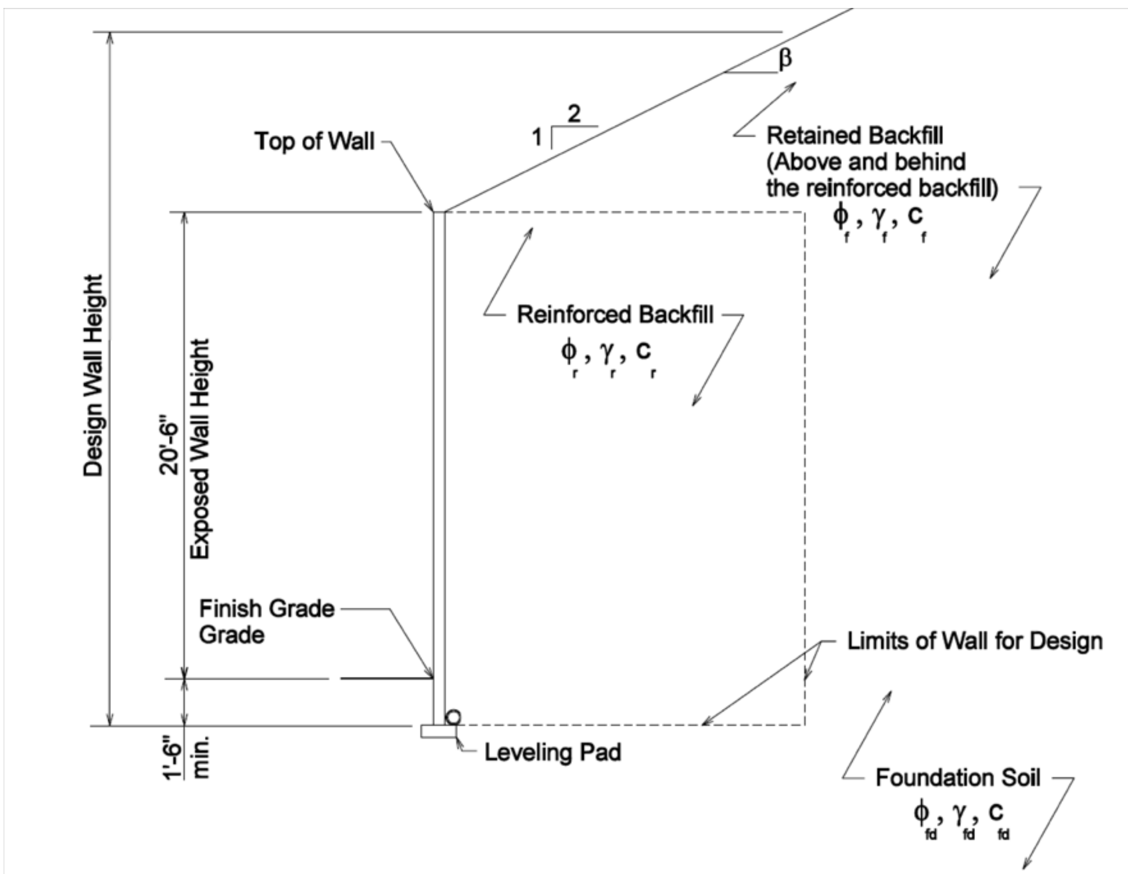

Figure E14-2.1-1
MSE Wall with Sloping Backfill

Wall Geometry

$H_e = 20.5$ Exposed wall height, ft

$H = H_e + 1.5$ Design wall height, ft (assume 1.5 ft wall embedment)

$\theta = 90\ \mathrm{deg}$ Angle of back face of wall to horizontal

$\beta = 26.565\ \mathrm{deg}$ Inclination of ground slope behind face of wall (2H:1V)

E14-2.2 Design Parameters

Project Parameters

$Design\_Life = 75$ Wall design life, years (min) **LRFD [11.5.1]**

Soil Properties (From Geotechnical Site Investigation Report)

Designer to determine if long-term or short-term soil strength parameters govern external stability.

Reinforced Backfill Soil Design Parameters

$\phi_r = 30\ \mathrm{deg}$ Angle of internal friction **LRFD [11.10.5.1]**

$\gamma_r = 0.120$ Unit of weight, kcf

$c_r = 0$ Cohesion, psf

Retained Backfill Soil Design Parameters

$\phi_f = 29\ \mathrm{deg}$ Angle of internal friction

$\gamma_f = 0.120$ Unit of weight, kcf

$c_f = 0$ Cohesion, psf

Foundation Soil Design Parameters

$\phi_{fd} = 31\mathrm{deg}$ Angle of internal friction

$\gamma_{fd} = 0.125$ Unit of weight, kcf

$c_{fd} = 0$ Cohesion, psf

Factored Bearing Resistance of Foundation Soil

$q_R = 10.0$ Factored resistance at the strength limit state, ksf

Note: The factored bearings resistance, q_R, was assumed to be given in the Site Investigation Report. If not provided q_R shall be determined by calculating the nominal bearing resistance, q_n, per **LRFD [Eq 10.6.3.1.2a-1]** and factored with the bearing resistance factor, ϕ_b, for MSE walls (i.e., $q_R = \phi_b q_n$).

Precast Concrete Panel Facing Parameters

$S_{vt} = 2.5$ Vertical spacing of reinforcement, ft

Note: vertical spacing should not exceed 2.7 ft without full scale test data
LRFD [11.10.6.2.1]

$w_p = 5.0$ Width of precast concrete panel facing, ft

$h_p = 5.0$ Height of precast concrete panel facing, ft

$t_p = 6.0$ Thickness of precast concrete panel facing, in

Soil Reinforcement Design Parameters

Galvanized steel ribbed strips Reinforcing type

$F_y = 65$ Reinforcing strip yield strength, ksi (Grade 65)

$b\_mm = 50$ Reinforcing strip width, mm

$$b = \frac{b\_mm}{25.4} \qquad \boxed{b = 1.97} \text{ in}$$

$E_{n\_mm} = 4$ Reinforcing strip thickness, mm

$$E_n = \frac{E_{n\_mm}}{25.4} \qquad \boxed{E_n = 0.16} \text{ in}$$

$Zinc = 3.4$ Zinc coating, mils (Minimum **LRFD [11.10.6.4.2a]**)

Live Load Surcharge Parameters

$SUR = 0.100$ Live load surcharge for walls without traffic, ksf (14.4.5.4.2)

Resistance Factors

$\phi_s = 1.00$	Sliding of MSE wall at foundation **LRFD [Table 11.5.6-1]**
$\phi_b = 0.65$	Bearing resistance **LRFD [Table 11.5.6-1]**
$\phi_t = 0.75$	Tensile resistance (steel strips) **LRFD [Table 11.5.6-1]**
$\phi_p = 0.90$	Pullout resistance **LRFD [Table 11.5.6-1]**

E14-2.3 Estimate Depth of Embedment and Length of Reinforcement

For this example it is assumed that global stability does not govern the required length of soil reinforcement.

Embedment Depth, d_e

Frost-susceptible material is assumed to be not present or that it has been removed and replaced with nonfrost susceptible material per **LRFD [11.10.2.2]**. There is also no potential for scour. Therefore, the minimum embedment, d_e, shall be the greater of 1.5 ft (14.6.4) or H/20 **LRFD [Table C11.10.2.2-1]**

Note: While AASHTO allows the d_e value of 1.0 ft on level ground, the embedment depth is limited to 1.5 ft by WisDOT policy as stated in Chapter 14.

$$\frac{H}{20} = 1.1 \quad \text{ft}$$

$$d_e = \max\left(\frac{H}{20}, 1.5\right) \qquad \boxed{d_e = 1.50} \ \text{ft}$$

Therefore, the initial design wall height assumption was correct.

$$H_e = 20.5 \quad \text{ft}$$

$$H = H_e + 1.5 \qquad \boxed{H = 22.00} \ \text{ft}$$

Length of Reinforcement, L

In accordance with **LRFD [11.10.2.1]** the minimum required length of soil reinforcement shall be the greater of 8 feet or 0.7H. Due to the sloping backfill surcharge and live load surcharge a longer reinforcement length of 0.9H will be used in this example. The length of reinforcement will be uniform throughout the entire wall height.

$0.9\,H = 19.8$ ft

$L_{user} = 20.0$ ft

$L = \max(8.0, 0.9\,H, L_{user})$ $\boxed{L = 20.00}$ ft

Height of retained fill at the back of the reinforced soil, h

$h = H + L\tan(\beta)$ $\boxed{h = 32.00}$ ft

E14-2.4 Permanent and Transient Loads

In this example, load types EV (vertical earth pressure), EH (horizontal earth pressure) and LS (live load surcharge) will be used as shown in Figure E14-2.4-1. Due to the relatively thin wall thickness the weight and width of the concrete facing will be ignored. Passive soil resistance will also be ignored.

E14-2.4.1 Compute Active Earth Pressure

Compute the coefficient of active earth pressure (k_a) using Coulomb Theory **LRFD [Eq 3.11.5.3-1]** with the wall backfill material interface friction angle, δ, set equal to β (i.e. $\delta=\beta$) **LRFD [11.10.5.2].** The retained backfill soil will be used (i.e., $k_a=k_{af}$)

$\phi_f = 29\,deg$

$\beta = 26.565\,deg$

$\theta = 90\,deg$

$\delta = \beta$

$$\Gamma = \left(1 + \sqrt{\frac{\sin(\phi_f + \delta)\sin(\phi_f - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right)^2 \qquad \boxed{\Gamma = 1.462}$$

$$k_{af} = \frac{\sin(\theta + \phi_f)^2}{\Gamma\sin(\theta)^2\sin(\theta - \delta)} \qquad \boxed{k_{af} = 0.585}$$

E14-2.4.2 Compute Unfactored Loads

The forces and moments are computed using Figure E14-2.4-1 by their appropriate LRFD load types **LRFD [Tables 3.4.1-1 and 3.4.1-2]**

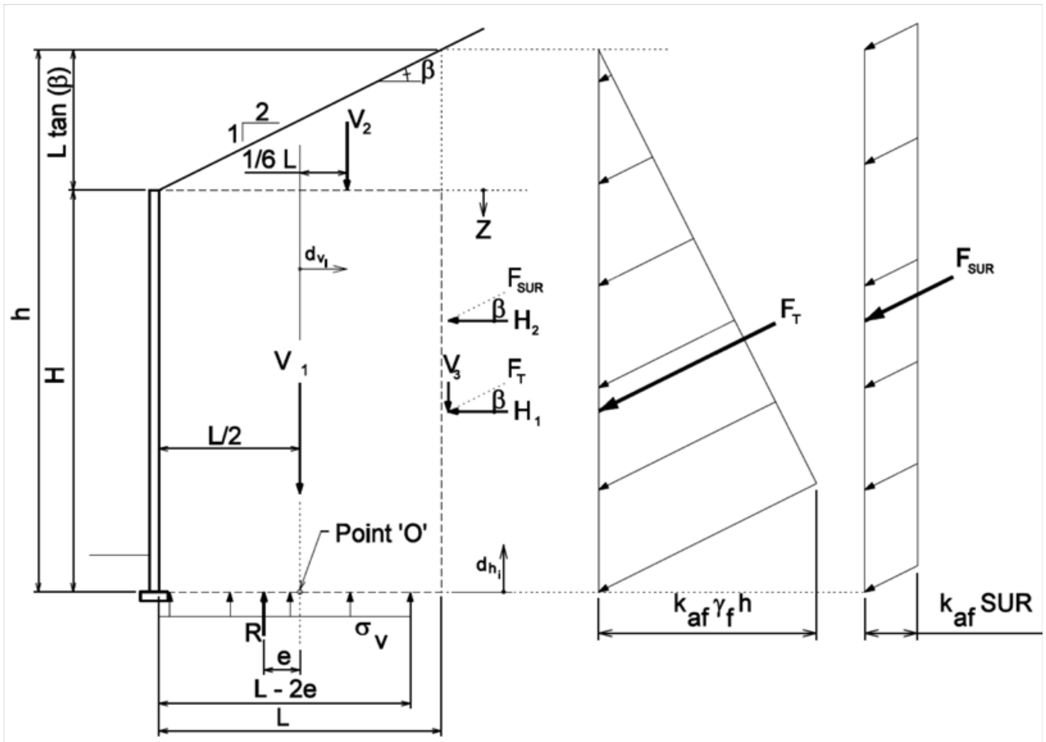

Figure E14-2.4-1
MSE Wall - External Stability

Active Earth Force Resultant, (kip/ft), F_T

$$F_T = \frac{1}{2}\,\gamma_f\,h^2\,k_{af}$$ Active earth force resultant (EH) $\boxed{F_T = 35.9}$

Live Load Surcharge Resultant, (kip/ft), F_{SUR}

$$F_{SUR} = SUR\ h\ k_{af}$$ Live load surcharge (LS) $\boxed{F_{SUR} = 1.9}$

Vertical Loads, (kip/ft), V_i

$$V_1 = \gamma_r\,H\,L$$ Soil backfill - reinforced soil (EV) $\boxed{V_1 = 52.8}$

$$V_2 = \frac{1}{2}\,\gamma_f\,L\,(L\,\tan(\beta))$$ Soil backfill - backslope (EV) $\boxed{V_2 = 12.0}$

$$V_3 = F_T\,\sin(\beta)$$ Active earth force resultant (vertical component - EH) $\boxed{V_3 = 16.1}$

Moments produced from vertical loads about Point 'O', (kip-ft/ft) MV_i

Moment Arm | Moment

$d_{v1} = 0$ $\boxed{d_{v1} = 0.0}$ $MV_1 = V_1\, d_{v1}$ $\boxed{MV_1 = 0.0}$

$d_{v2} = \frac{1}{6}L$ $\boxed{d_{v2} = 3.3}$ $MV_2 = V_2\, d_{v2}$ $\boxed{MV_2 = 40.0}$

$d_{v3} = \frac{L}{2}$ $\boxed{d_{v3} = 10.0}$ $MV_3 = V_3\, d_{v3}$ $\boxed{MV_3 = 160.7}$

Horizontal Loads, (kip/ft), H_i

$H_1 = F_T \cos(\beta)$ Active earth force resultant (horizontal component - EH) $\boxed{H_1 = 32.1}$

$H_2 = F_{SUR} \cos(\beta)$ Live load surcharge resultant (horizontal component - LS) $\boxed{H_2 = 1.7}$

Moments produced from horizontal loads about Point 'O', (kip-ft/ft), MH_i

Moment Arm | Moment

$d_{h1} = \frac{h}{3}$ $\boxed{d_{h1} = 10.7}$ $MH_1 = H_1\, d_{h1}$ $\boxed{MH_1 = 342.8}$

$d_{h2} = \frac{h}{2}$ $\boxed{d_{h2} = 16.0}$ $MH_2 = H_2\, d_{h2}$ $\boxed{MH_2 = 26.8}$

Summary of Unfactored Forces & Moments:

Load			Moment Arm		Moment		LRFD Load Type
Item	Description	Value (kip/ft)	Item	Value (ft)	Item	Value (kip-ft/ft)	
V_1	Soil backfill	52.80	d_{v1}	0.0	MV_1	0.0	EV
V_2	Soil backfill	12.00	d_{v2}	3.3	MV_2	40.0	EV
V_3	Active earth pressure	16.10	d_{v3}	10.0	MV_3	160.7	EH

Table E14-2.4-1
Unfactored Vertical Forces & Moments

Load			Moment Arm		Moment		LRFD Load Type
Item	Description	Value (kip/ft)	Item	Value (ft)	Item	Value (kip-ft/ft)	
H_1	Active earth pressure	32.1	d_{h1}	10.7	MH_1	342.8	EH
H_2	Live load surcharge	1.70	d_{h2}	16.0	MH_2	26.8	LS

Table E14-2.4-2
Unfactored Horizontal Forces & Moments

E14-2.4.3 Summarize Applicable Load and Resistance Factors

Maximum and minimum load factors shall be used to determine the extreme load effects. WisDOT's policy is to set all load modifiers to one ($n = 1.0$). Factored loads and moments for each limit state are calculated by applying the appropriate load factors **LRFD [Tables 3.4.1-1 and 3.4.1-2]**. The following load combinations will be checked in this example:

Load Combination Limit State	EV	LS	EH
Strength Ia (minimum)	$\gamma_{EVmin} = 1.00$	$\gamma_{LSmin} = 1.75$	$\gamma_{EHmin} = 0.90$
Strength Ib (maximum)	$\gamma_{EVmax} = 1.35$	$\gamma_{LSmax} = 1.75$	$\gamma_{EHmax} = 1.50$
Service I (max/min)	$\gamma_{EV} = 1.00$	$\gamma_{LS} = 1.00$	$\gamma_{EH} = 1.00$

Load Combination	γ_{EV}	γ_{LS_V}	γ_{LS_H}	γ_{EH}	Application
Strength Ia	1.00	-	1.75	1.50	Sliding, Eccentricity
Strength Ib	1.35	1.75	1.75	1.50	Bearing, T_{max}
Service I	1.00	1.00	1.00	1.00	Pullout (σ_v)

Table E14-2.4-3
Unfactored Horizontal Forces & Moments

Load Combination Assumptions:

- Live load surcharge stabilizing loads (if applicable) are ignored for overturning and sliding analyses. Live load surcharge is used to compute maximum bearing pressure, wall strength and overall (global) stability.
- Minimum horizontal earth pressure ,$\gamma_{EH(min)}$ = 0.9, will not control in this example based on B/H and lateral load inclination, but should be checked.
- Component load factors shall remain consistent throughout calculations. For example, the active earth force resultant (F_T) can be broken into component forces of either $V_3\gamma_{EH(max)}$ and $H_1\gamma_{EH(max)}$ or $V_3\gamma_{EH(min)}$ and $H_1\gamma_{EH(min)}$, not $V_3\gamma_{EH(min)}$ and $H_1\gamma_{EH(max)}$.
- T_{max1} (Pullout) is calculated without live load and T_{max2} (Rupture) is calculated with live load.

The loads discussed and tabulated previously can now be factored by the appropriate load factors and combined to determine the governing limit states for each design check.

E14-2.4.3 Compute Factored Loads and Moments

Unfactored loads by load type (kip/ft)

$$V_{EV} = V_1 + V_2 \qquad \boxed{V_{EV} = 64.8}$$

$$V_{EH} = V_3 \qquad \boxed{V_{EH} = 16.1}$$

$$H_{EH} = H_1 \qquad \boxed{H_{EH} = 32.1}$$

$$H_{LS} = H_2 \qquad \boxed{H_{LS} = 1.7}$$

Unfactored moments by load type (kip-ft/ft)

$$M_{EV} = MV_1 + MV_2 \qquad \boxed{M_{EV} = 40.0}$$

$$M_{EH1} = MV_3 \qquad \boxed{M_{EH1} = 160.7}$$

$$M_{EH2} = MH_1 \qquad \boxed{M_{EH2} = 342.8}$$

$$M_{LS2} = MH_2 \qquad \boxed{M_{LS2} = 26.8}$$

Factored vertical loads by limit state (kip/ft)

$$V\_Ia = n\left(1.00V_{EV} + 1.50\ V_{EH}\right) \qquad \boxed{V\_Ia = 88.9}$$

$$V\_Ib = n\left(1.35V_{EV} + 1.50\ V_{EH}\right) \qquad \boxed{V\_Ib = 111.6}$$

$$V\_Ser = n\left(1.00V_{EV} + 1.00\ V_{EH}\right) \qquad \boxed{V\_Ser = 80.9}$$

Factored horizontal loads by limit state (kip/ft)

$$H\_Ia = n\left(1.75H_{LS} + 1.50H_{EH}\right) \qquad \boxed{H\_Ia = 51.1}$$

$$H\_Ib = n\left(1.75H_{LS} + 1.50H_{EH}\right) \qquad \boxed{H\_Ib = 51.1}$$

$$H\_Ser = n\left(1.00H_{LS} + 1.00H_{EH}\right) \qquad \boxed{H\_Ser = 33.8}$$

Factored moments produced by vertical Loads by limit state (kip-ft/ft)

$$MV\_Ia = n\left(1.00M_{EV} + 1.50\ M_{EH1}\right) \qquad \boxed{MV\_Ia = 281.0}$$

$$MV\_Ib = n\left(1.35M_{EV} + 1.50\ M_{EH1}\right) \qquad \boxed{MV\_Ib = 295.0}$$

$$MV\_Ser = n\left(1.00M_{EV} + 1.00\ M_{EH1}\right) \qquad \boxed{MV\_Ser = 200.7}$$

Factored moments produced by horizontal loads by limit state (kip-ft/ft)

$$MH\_Ia = n\left(1.75M_{LS2} + 1.50\ M_{EH2}\right) \qquad \boxed{MH\_Ia = 561.1}$$

$$MH\_Ib = n\left(1.75M_{LS2} + 1.50\ M_{EH2}\right) \qquad \boxed{MH\_Ib = 561.1}$$

$$MH\_Ser = n\left(1.00M_{LS2} + 1.00\ M_{EH2}\right) \qquad \boxed{MH\_Ser = 369.6}$$

Summary of Factored Forces & Moments:

Load Combination	Vert. Loads V (kips/ft)	Moments MV (kip-kip/ft)	Horiz. Loads H (kips/ft)	Moments MH (kip-kip/ft)
Strength Ia	88.9	281.0	51.1	561.1
Strength Ib	111.6	295.0	51.1	561.1
Service I	80.9	200.7	33.8	369.6

Table E14-2.4-4
Summary of Factored Loads & Moments

E14-2.5 Evaluate External Stability of MSE Wall

Three potential external failure mechanisms will be considered in this example (sliding at the base, limiting eccentricity and bearing resistance). Global (overall) stability requirements are assumed to have been satisfied in prior calculations. Design calculations will be carried out for the governing limit states only.

E14-2.5.1 Sliding Resistance at Base of MSE Wall

The following calculations are based on **Strength Ia**:

Factored Sliding Force

$R_u = H\_Ia$ $\boxed{R_u = 51.14}$ kip/ft

Sliding Resistance

To compute the coefficient of sliding friction for discontinuous reinforcement use the lesser friction angle of the reinforced back fill, ϕ_r , or foundation soil, ϕ_{fd}, **LRFD [11.10.5.3]**.

$\phi_\mu = \min(\phi_r, \phi_{fd})$ $\boxed{\phi_\mu = 30\ \text{deg}}$

$\mu = \tan(\phi_\mu)$ $\boxed{\mu = 0.577}$

$V\_Ia = 88.9$ Factored vertical load, kip/ft

$V_{Nm} = \mu\ V\_Ia$ $\boxed{V_{Nm} = 51.3}$ kip/ft

$\phi_s = 1.0$

$R_R = \phi_s\ V_{Nm}$ $\boxed{R_R = 51.33}$ kip/ft

<u>Capacity:Demand Ratio (CDR)</u>

$CDR_{Sliding} = \dfrac{R_R}{R_u}$ $\boxed{CDR_{Sliding} = 1.00}$

Is the $CDR \geq 1.0$? $\boxed{check = \text{"OK"}}$

E14-2.5.2 Limiting Eccentricity at Base of MSE Wall

The location of the resultant of the reaction forces is limited to the middle one-half of the base width for a soil foundation (i.e., e_{max} = L/4) **LRFD [11.6.3.3]**. The following calculations are based on **Strength Ia**:

Maximum eccentricity

$$e_{max} = \frac{L}{4}$$ $\boxed{e_{max} = 5.00}$ ft

Compute wall eccentricity (distance from Point 'O' in Figure E14-2.4-1)

$\Sigma M_R = MV\_Ia$ Summation of resisting moments for Strength Ia

$\Sigma M_O = MH\_Ia$ Summation of overturning moments for Strength Ia

$\Sigma V = V\_Ia$ Summation of vertical loads for Strength Ia

$\boxed{\Sigma M_R = 281.0}$ kip-ft/ft

$\boxed{\Sigma M_O = 561.1}$ kip-ft/ft

$\boxed{\Sigma V = 88.9}$ kip/ft

$$e = \frac{\Sigma M_O - \Sigma M_R}{\Sigma V}$$ $\boxed{e = 3.15}$ ft

<u>Capacity:Demand Ratio (CDR)</u>

$$CDR_{Eccentricity} = \frac{e_{max}}{e}$$ $\boxed{CDR_{Eccentricity} = 1.59}$

Is the $CDR \geq 1.0$? $\boxed{check = \text{"OK"}}$

E14-2.5.3 Bearing Resistance at base of MSE Wall

The following calculations are based on **Strength Ib**:

Compute wall eccentricity (distance from Point 'O' in Figure E14-2.4-1)

$\Sigma M_R = MV\_Ib$ Summation of resisting moments for Strength Ib

$\Sigma M_O = MH\_Ib$ Summation of overturning moments for Strength Ib

$\Sigma V = V\_Ib$ Summation of vertical loads for Strength Ib

$\boxed{\Sigma M_R = 295.0}$ kip-ft/ft

$\boxed{\Sigma M_O = 561.1}$ kip-ft/ft

$\boxed{\Sigma V = 111.6}$ kip/ft

$$e = \frac{\Sigma M_O - \Sigma M_R}{\Sigma V}$$

$\boxed{e = 2.38}$ ft

Compute the ultimate bearing stress

$\sigma_{\cdot V}$ = Ultimate bearing stress
L = Bearing length
e = Eccentricity (resultant produced by extreme bearing resistance loading)

Note: For the bearing resistance calculations the effective bearing width, B' = L-2e, is used instead of the actual width. Also, when the eccentricity, e, is negative: B'=L. The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**.

$$\sigma_V = \frac{\Sigma V}{L - 2\ e}$$

$\boxed{\sigma_V = 7.33}$ ksf/ft

Factored bearing resistance

$\boxed{q_R = 10.00}$ ksf/ft

Capacity:Demand Ratio (CDR)

$$CDR_{Bearing} = \frac{q_R}{\sigma_V}$$

$\boxed{CDR_{Bearing} = 1.37}$

Is the $CDR \geq 1.0$?

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

E14-2.6 Evaluate Internal Stability of MSE Wall

Note: MSE walls are a proprietary wall system and the internal stability computations shall be performed by the wall supplier.

Internal stability shall be checked for 1) pullout and 2) rupture in accordance with **LRFD [11.10.6]**. The factored tensile load, T_{max}, is calculated twice for internal stability checks for vertical stress (σ_V) calculations. For pullout T_{max1} is determined by excluding live load surcharge. For rupture T_{max2} is determined by including live load surcharge. In this example, the maximum reinforcement loads are calculated using the Simplified Method.

The location of the potential failure surface for a MSE wall with metallic strip or grid reinforcements (inextensible) is shown in Figure E14-2.6-1.

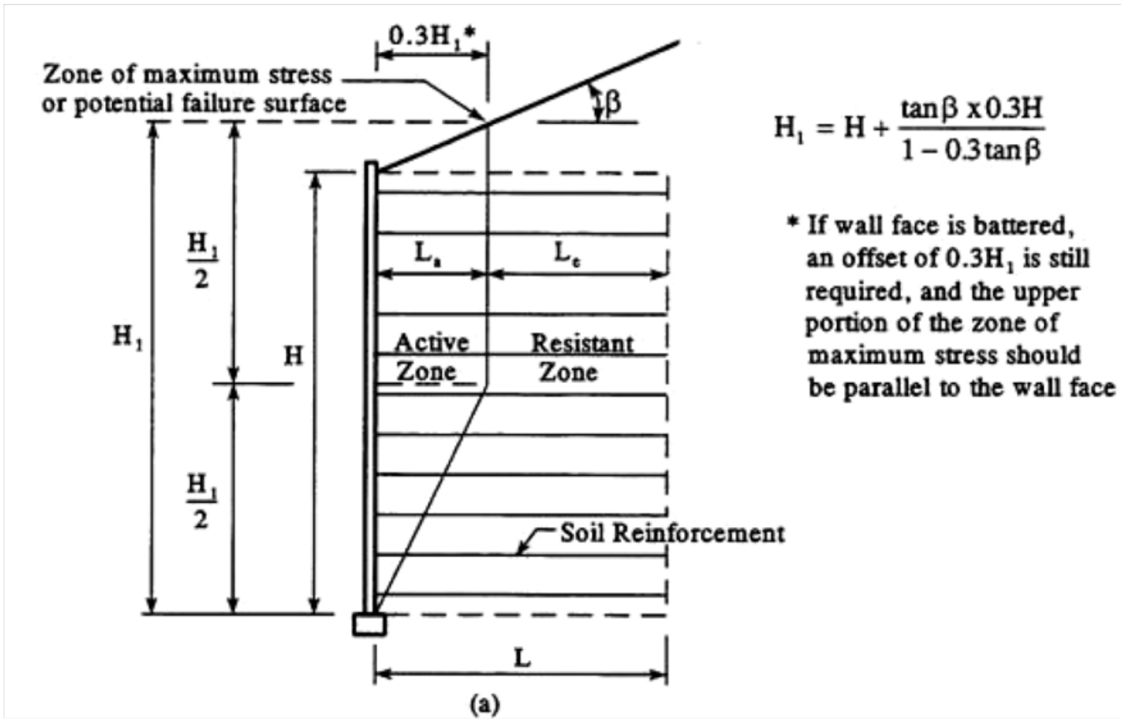

Figure E14-2.6-1
MSE Wall - Internal Stability (Inextensible Reinforcement)
FHWA [Figure 4-9]

E14-2.6.1 Establish the Vertical Layout of Soil Reinforcement

Soil reinforcement layout is shown in Table E14-2.6-1. The location of the reinforcement levels corresponds to the vertical depth, Z, into the reinforced soil. The vertical layout was determined by the industry practice of 2.5 ft vertical spacing for steel ribbed strip reinforcement. The top level vertical spacing was adjusted to fit the height of the wall. Computations for determining the maximum tension, T_{max}, at Z = 8.25 ft are as follows:

$Layer = 4$ Layer of reinforcement (from top)

$Z = 8.25$ Depth below top of wall, ft

$S_{vt} = 2.5$ Vertical spacing of reinforcement, ft

$w_p = 5.00$ Width of precast concrete panel facing, ft

Calculate the upper and lower tributary depths based on the reinforcement vertical spacing

$$Z_{neg} = Z - \frac{S_{vt}}{2} \qquad \boxed{Z_{neg} = 7.0} \text{ ft}$$

$$Z_{pos} = Z + \frac{S_{vt}}{2} \qquad \boxed{Z_{pos} = 9.5} \text{ ft}$$

Layer	Z (ft)	Z^- (ft)	Z^+ (ft)	S_{vt} (ft)
1	0.75	0	0.75+0.5(3.25-0.75)= 2.0	2.00
2	3.25	3.25-0.5(3.25-0.75)= 2.0	3.25+0.5(5.75-3.25)= 4.5	2.50
3	5.75	5.75-0.5(5.75-3.25)= 4.5	5.75+0.5(8.25-5.75)= 7.0	2.50
4	**8.25**	**8.25-0.5(8.25-5.75)= 7.0**	**8.25+0.5(10.75-8.25)= 9.5**	**2.50**
5	10.75	10.75-0.5(10.75-8.25)= 9.5	10.75+0.5(13.25-10.75)= 12.0	2.50
6	13.25	13.25-0.5(13.25-10.75)= 12.0	13.25+0.5(15.75-13.25)= 14.5	2.50
7	15.75	15.75-0.5(15.75-13.25)= 14.5	15.75+0.5(18.25-15.75)= 17.0	2.50
8	18.25	18.25-0.5(18.25-15.75)= 17.0	18.25+0.5(20.75-18.25)= 19.5	2.50
9	20.75	20.75-0.5(20.75-18.25)= 19.5	22	2.50

Table E14-2.6-1
Summary of Computations for Reinforcement Spacing, S_{vt}

E14-2.6.2 Compute Horizontal Stress and Maximum Tension, T_{max}

Factored horizontal stress

$$\sigma_H = \gamma_P (\sigma_v k_r + \Delta\sigma_H)$$ **LRFD [Equation 11.10.6.2.1-1]**

γ_p = Load factor for vertical earth pressure (γ_{EVmax})

k_r = Horizontal pressure coefficient

σ_v = Pressure due to gravity and surcharge for pullout, T_{max1} ($\gamma_r Z_{trib} + \sigma_2$)

σ_v = Pressure due to gravity and surcharge for pullout resistance ($\gamma_r Z_{p\text{-}PO}$)

σ_v = Pressure due to gravity and surcharge for rupture, T_{max2} ($\gamma_r Z_{trib} + \sigma_2 + q$)

$\Delta\sigma_H$ = Horizontal pressure due to concentrated horizontal surcharge load

Z = Reinforcement depth for max stress Figure E14-2.6-2

Z_p = Depth of soil at reinforcement layer potential failure plane

$Z_{p\text{-}ave}$ = Average depth of soil at reinforcement layer in the effective zone

σ_2 = Equivalent uniform stress from backslope $(0.5(0.7)L\tan\beta)\gamma_f$

q = Surcharge load ($q = SUR$), ksf

To compute the lateral earth pressure coefficient, k_r, a k_a multiplier is used to determine k_r for each of the respective vertical tributary spacing depths (Z_{pos}, Z_{neg}). The k_a multiplier is determined using Figure E14-2.6-2. To calculate k_a it is assumed that $\delta=\beta$ and $\beta = 0$; thus, $k_a=\tan^2(45-\phi_f/2)$ **LRFD [Equation C11.10.6.2.1-1]**

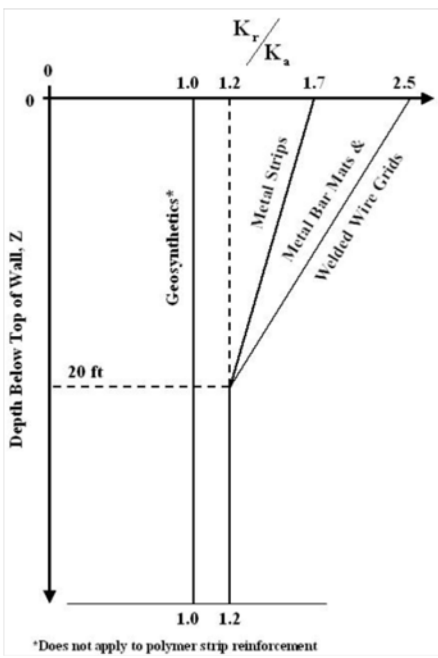

Figure E14-2.6-2
k_r/k_a Variation with MSE Wall Depth
FHWA [Figure 4-10]

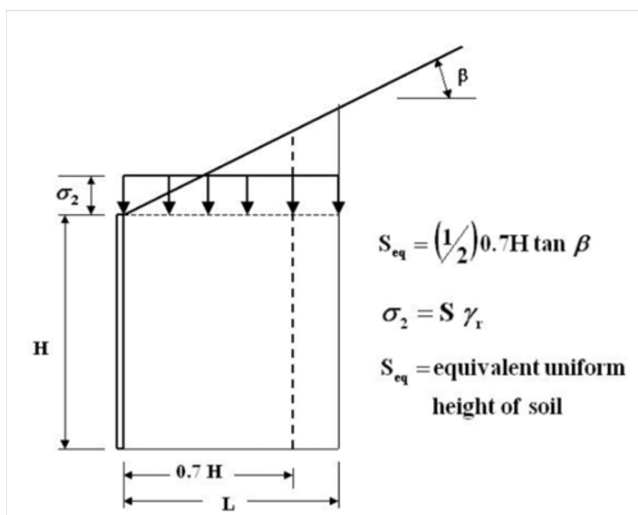

Figure E14-2.6-3
Calculation of Vertical Stress
FHWA [Figure 4-11]

Calculate the coefficient of active earth pressure, k_a

$\phi_f = 29\,deg$

$$k_a = \tan\left(45\ deg - \frac{\phi_f}{2}\right)^2$$

$\boxed{k_a = 0.347}$

Compute the internal lateral earth pressure coefficient limits based on applying a k_a multiplier as shown in Figure E14-2.6-2. For inextensible steel ribbed strips the k_a multiplier decreases linearly from the top of the reinforced soil zone to a depth of 20 ft. Thus, the k_a multiplier will vary from 1.7 at Z=0 ft to 1.2 at Z=20 ft. To compute k_r apply these values to the coefficient of active earth pressure.

$k_{r\_0ft} = 1.7\ k_a$ $\boxed{k_{r\_0ft} = 0.590}$

$k_{r\_20ft} = 1.2\ k_a$ $\boxed{k_{r\_20ft} = 0.416}$

Compute the internal lateral earth pressure coefficients, k_r, for each of the respective tributary depths. Since both depths, Z_{neg} and Z_{pos}, are less than 20 ft k_r will be interpolated at their respective depths

$$k_{r\_neg} = k_{r\_20ft} + \frac{(20 - Z_{neg})(k_{r\_0ft} - k_{r\_20ft})}{20}$$ $\boxed{k_{r\_neg} = 0.529}$

$$k_{r\_pos} = k_{r\_20ft} + \frac{(20 - Z_{pos})(k_{r\_0ft} - k_{r\_20ft})}{20}$$ $\boxed{k_{r\_pos} = 0.507}$

Compute effective (resisting) length, L_e

$Z = 8.25$ ft Refer to Figure E14-2.6-1. ($\Delta H = H_1 - H$)

$H = 22.0$ ft

$L = 20$ ft

$$\Delta H = \frac{\tan(\beta)\,(0.3\ H)}{1 - 0.3\tan(\beta)}$$ $\boxed{\Delta H = 3.88}$ ft

$H_1 = H + \Delta H$ $\boxed{H_1 = 25.9}$ ft

$$L_a = \begin{cases} 0.3\ H_1 & \text{if } Z \le \dfrac{H_1}{2} - \Delta H \\ \dfrac{H - Z}{\frac{H_1}{2}}(0.3\ H_1) & \text{otherwise} \end{cases}$$ $\boxed{L_a = 7.76}$ ft

$L_e = \max(L - L_a, 3)$ $\boxed{L_e = 12.24}$ ft

Note: L_e shall be greater than or equal to 3 feet **LRFD [11.10.6.3.2]**

E14-2.6.3 Establish Nominal and Factored Pullout Resistance of Soil Reinforcement

Compute the factored horizontal stress, σ_H, at Z by averaging the upper and lower tributary values (Z_{neg} and Z_{pos}). Since there is no horizontal stresses from concentrated dead loads values $\Delta\sigma_H$ is set to zero.

$$\sigma_H = \gamma_{EVmax}\,(\gamma_r Z_{trib} + \sigma_2)k_r$$

Surcharge loads

$$\sigma_2 = \frac{1}{2}\,0.7\,H\,\tan(\beta)\,\gamma_f \qquad \boxed{\sigma_2 = 0.46} \text{ ksf/ft}$$

Horizontal stress at Z_{neg} and Z_{pos}

$$\sigma_{H\_neg} = \gamma_{EVmax}\left(\gamma_r Z_{neg} + \sigma_2\right) k_{r\_neg} \qquad \boxed{\sigma_{H\_neg} = 0.93} \text{ ksf/ft}$$

$$\sigma_{H\_pos} = \gamma_{EVmax}\left(\gamma_r Z_{pos} + \sigma_2\right) k_{r\_pos} \qquad \boxed{\sigma_{H\_pos} = 1.10} \text{ ksf/ft}$$

Horizontal stress at Z

$$\sigma_H = 0.5\left(\sigma_{H\_pos} + \sigma_{H\_neg}\right) \qquad \boxed{\sigma_H = 1.01} \text{ ksf/ft}$$

Compute the maximum tension, T_{max1} , at Z

$$A_{trib} = S_{vt}\,w_p \qquad \boxed{A_{trib} = 12.50} \text{ ft}^2$$

$$T_{max1} = \sigma_H\,A_{trib} \qquad \boxed{T_{max1} = 12.67} \text{ kip/strip}$$

Compute effective vertical stress for pullout resistance, σ_V

$$Z_{p\_PO} = Z + 0.5\tan(\beta)\left(L_a + L\right) \qquad \boxed{Z_{p\_PO} = 15.2} \text{ ft}$$

$\gamma_{EV} = 1.00$ Unfactored vertical stress for pullout resistance **LRFD [11.10.6.3.2]**

$$\sigma_V = \gamma_{EV}\,\gamma_r\,Z_{p\_PO} \qquad \boxed{\sigma_V = 1.82} \text{ ksf}$$

Compute pullout resistance factor, F*

The coefficient of uniformity, C_u, shall be computed based on backfill gradations D_{60}/D_{10}. If the backfill material is unknown at the time of design a conservative assumption of C_u=4 should be assumed **LRFD [11.10.6.3.2]**.

The pullout resistance factor, F*, for inextensible steel ribbed strips decreases linearly from the top of the intersection of the failure plane with the top of the reinforced soil zone. Thus, F* will vary from 1.2+log C_u (<= 2.0) at Z=0 ft to tan(ϕ_r) at Z=20 ft. Since no product-specific pullout test data is provided at the time of design the default value for F* will be used as provided by **LRFD [Figure 11.10.6.3.2-1]**.

$C_u = 4$ Coefficient of uniformity (C_u=4 default value) **LRFD [11.10.6.3.2]**

$F'_{0ft} = \min(2.00, 1.2 + \log(C_u))$ $\boxed{F'_{0ft} = 1.80}$

$F'_{20ft} = \tan(\phi_r)$ $\boxed{F'_{20ft} = 0.58}$

$$F' = \begin{cases} F'_{20ft} + \dfrac{20.0 - Z}{20}(F'_{0ft} - F'_{20ft}) & \text{if } Z \le 20.0 \\ \tan(\phi_r) & \text{otherwise} \end{cases}$$

$\boxed{F' = 1.30}$

Compute nominal pullout resistance, P_r

$\alpha = 1.0$ Scale effect correction factor (steel reinforcement $\alpha = 1.0$ default value) **LRFD [Table 11.10.6.3.2-1]**

$C = 2$ Overall reinforcement surface area geometry factor (strip reinforcement $C = 2.0$) **LRFD [11.10.6.3.2]**

$R_c = 1$ Reinforcement coverage ratio (continuous reinforcement $R_c = 1.0$) **LRFD [11.10.6.4]**

Note: Using strips are considered discontinuous, however the nominal pullout resistance is based on the actual strip width, rather than a unit width, the reinforcement coverage ratio is 1.

$P_r = F' \, \alpha \, \sigma_v \, C \, R_c \, L_e \, b \, \dfrac{1}{12}$ $\boxed{P_r = 9.49}$ kip/strip

Compute factored pullout resistance, P_{rr}

$\phi_p = 0.9$

$P_{rr} = \phi_p \, P_r$ $\boxed{P_{rr} = 8.54}$ kip/strip

Determine number of soil reinforcing strips based on pullout resistance, N_p

$N_p = \dfrac{T_{max1}}{P_{rr}}$ $\boxed{N_p = 1.48}$ strips

E14-2.6.4 Establish Nominal and Factored Long-term Tensile Resistance of Soil Reinforcement

Compute the factored horizontal stress, σ_H

$$\sigma_H = \gamma_{EVmax} (\gamma_r Z + \sigma_2 + q) k_r$$

Surcharge loads

$\boxed{\sigma_2 = 0.46}$ ksf/ft

$\boxed{q = 0.10}$ ksf/ft

Horizontal stress at Z_{neg} and Z_{pos}

$\sigma_{H\_neg} = \gamma_{EVmax} \left(\gamma_r Z_{neg} + \sigma_2 + q\right) k_{r\_neg}$ $\boxed{\sigma_{H\_neg} = 1.00}$ ksf/ft

$\sigma_{H\_pos} = \gamma_{EVmax} \left(\gamma_r Z_{pos} + \sigma_2 + q\right) k_{r\_pos}$ $\boxed{\sigma_{H\_pos} = 1.17}$ ksf/ft

Horizontal stress at Z

$\sigma_H = 0.5\left(\sigma_{H\_pos} + \sigma_{H\_neg}\right)$ $\boxed{\sigma_H = 1.08}$ ksf/ft

Compute the maximum tension, T_{max} , at Z

$A_{trib} = S_{vt}\, w_p$ $\boxed{A_{trib} = 12.50}$ ft^2

$T_{max2} = \sigma_H\, A_{trib}$ $\boxed{T_{max2} = 13.55}$ kip/strip

E_c = thickness of metal reinforcement at end of service life (mil)
E_n = nominal thickness of steel reinforcement at construction (mil)
E_s = sacrificial thickness of metal lost by corrosion during service life of structure (mil)
b = width of metal reinforcement

$F_y = 65$ Reinforcing strip yield strength, ksi

$\phi_t = 0.75$ Tensile resistance (steel strip)

$E_n = 0.16$ Reinforcing strip thickness, in

$b = 1.97$ Reinforcing strip width, in

$Zinc = 3.4$ Galvanized coating, mils

Compute the design cross-sectional area of the reinforcement after sacrificial thicknesses have been accounted for during the wall design life per **LRFD [11.10.6.4.2a]**. The zinc coating life shall be calculated based on 0.58 mil/yr loss for the first 2 years and 0.16 mil/yr thereafter. After the depletion of the zinc coating, the steel design life is calculated and used to determine the sacrificial steel thickness after the steel design life. The sacrificial thickness of steel is based on 0.47 mil/yr/side loss.

Design_Life = Coating_Life + Steel_Design_Life = 75 years

$$Coating\_Life = 2 + \frac{Zinc - 2\ 0.58}{0.16} \qquad \boxed{Coating\_Life = 16.0} \text{ years}$$

$$Steel\_Design\_Life = Design\_Life - Coating\_Life \qquad \boxed{Steel\_Design\_Life = 59} \text{ years}$$

$$E_s = \left(\frac{0.47}{1000}\right) Steel\_Design\_Life\ (2) \qquad \boxed{E_s = 0.055} \text{ in}$$

$$E_c = E_n - E_s \qquad \boxed{E_c = 0.102} \text{ in}$$

$$Design\_Strip\_Area = E_c\ b \qquad \boxed{Design\_Strip\_Area = 0.201} \text{ in}^2$$

Compute the Factored Tensile Resistance, T_r

$$T_n = F_y\ Design\_Strip\_Area \qquad \boxed{T_n = 13.05} \text{ kip/strip}$$

$$T_r = \phi_t\ T_n \qquad \boxed{T_r = 9.79} \text{ kip/strip}$$

Determine the number of soil reinforcing strips based on tensile resistance, N_t

$$N_t = \frac{T_{max2}}{T_r} \qquad \boxed{N_t = 1.38} \text{ strips}$$

E14-2.6.5 Establish Number of Soil Reinforcing Strips at Z

$N_p = 1.48$ Based on pullout resistance, strips

$N_t = 1.38$ Based on tensile resistance, strips

Required number of strip reinforcements for each panel width (round up), N_g

$$N_g = ceil(max(N_t, N_p)) \qquad \boxed{N_g = 2} \text{ strips}$$

Calculate the horizontal spacing of reinforcement, S_h, at Z by dividing the panel width by the required number of strip reinforcements N_g.

$$S_h = \frac{w_p}{N_g} \qquad \boxed{S_h = 2.50} \text{ ft}$$

Note: The typical horizontal reinforcement spacing, S_h, will be provided at 2.5 ft. This will also be the maximum allowed spacing while satisfying the maximum spacing requirement of 2.7 ft. If the wall requires additional reinforcement the vertical spacing will be maintained and adjustments will be made to the horizontal spacing accordingly.

E14-2.7 Summary of Results

E14-2.7.1 Summary of External Stability

Based on the defined project parameters, embedment depth and length of reinforcement the following external stability checks have been satisfied:

External Check	CDR
Sliding	1.00
Eccentricity	1.59
Bearing	1.37

Table E14-2.7-1
Summary of External Stability Computations

E14-2.7.2 Summary of Internal Stability

Computations for the required number of strip reinforcements at each level is presented in **Table E14-2.7-2**.

		Pullout			Rupture						
Layer	Z	σ_H	T_{max1}	P_{rr}	σ_H	T_{max2}	T_r	N_p	N_t	N_g	S_h
1	0.75	0.46	4.55	5.86	0.53	5.34	9.79	0.78	0.54	2	2.50
2	3.25	0.64	8.05	7.08	0.72	9.00	9.79	1.14	0.92	2	2.50
3	5.75	0.84	10.47	7.98	0.91	11.38	9.79	1.31	1.16	2	2.50
4	**8.25**	**1.01**	**12.67**	**8.54**	**1.08**	**13.55**	**9.79**	**1.48**	**1.38**	**2**	**2.50**
5	10.75	1.17	14.65	9.37	1.24	15.49	9.79	1.56	1.58	2	2.50
6	13.25	1.31	16.42	10.13	1.38	17.22	9.79	1.62	1.76	2	2.50
7	15.75	1.44	17.96	10.46	1.50	18.73	9.79	1.72	1.91	2	2.50
8	18.25	1.54	19.29	10.25	1.60	20.01	9.79	1.88	2.04	3	1.67
9	20.75	1.67	20.84	10.22	1.72	21.55	9.79	2.04	2.20	3	1.67

Table E14-2.7-2
Summary of Internal Stability Computation for Strength I Load Combinations

E14-2.7.3 Element Facings and Drainage Design

The design of element facings will not be examined in this example, but shall be considered in the design. This is to be performed by the wall supplier. This includes, but is not limited to, the structural integrity of the concrete face panels, connections, joint widths, differential settlements and the design of bearing pads used to prevent or minimize point loadings or stress concentrations and to accommodate for small vertical deformations of the panels.

Drainage requirements shall be investigated and detailed accordingly. In this example drainage requirements are meet by including a wrapped pipe underdrain behind the retaining wall as shown in Figure E14-2.8-1.

E14-2.8 Final MSE Wall Schematic

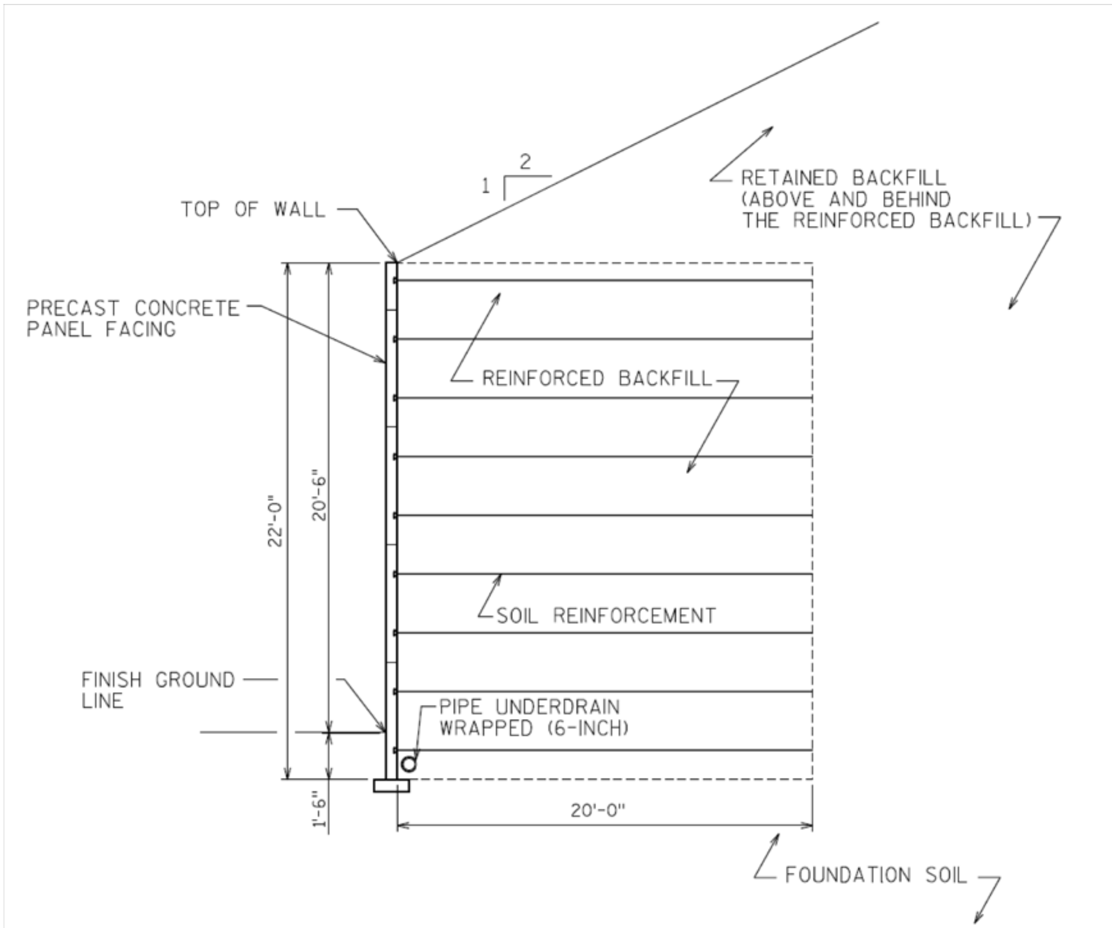

Figure E14-2.8-1
MSE Wall Schematic

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E14-3 Modular Block Facing Geogrid Reinforced MSE Wall, LRFD

General

This example shows design calculations for MSE wall with modular block facings conforming to the LRFD Bridge Design Specifications and the WisDOT Bridge Manual. *(Example is current through LRFD Fifth Edition - 2010)*

Sample design calculations for external stability (sliding, eccentricity and bearing) and internal stability (soil reinforcement stress and pullout) will be presented. The overall stability, settlement and connection calculations will not be shown in this example, but are required.

Design steps presented in 14.6.3.3 are used for the wall design.

E14-3.1 Establish Project Requirements

The following MSE wall shall have compacted freely draining soil in the reinforced zone and will be reinforced with geosynthetic (extensible) strips as shown in Figure E14-3.1-1. External stability is the designer's (WisDOT/Consultant) responsibility and internal stability and structural components are the contractors responsibility.

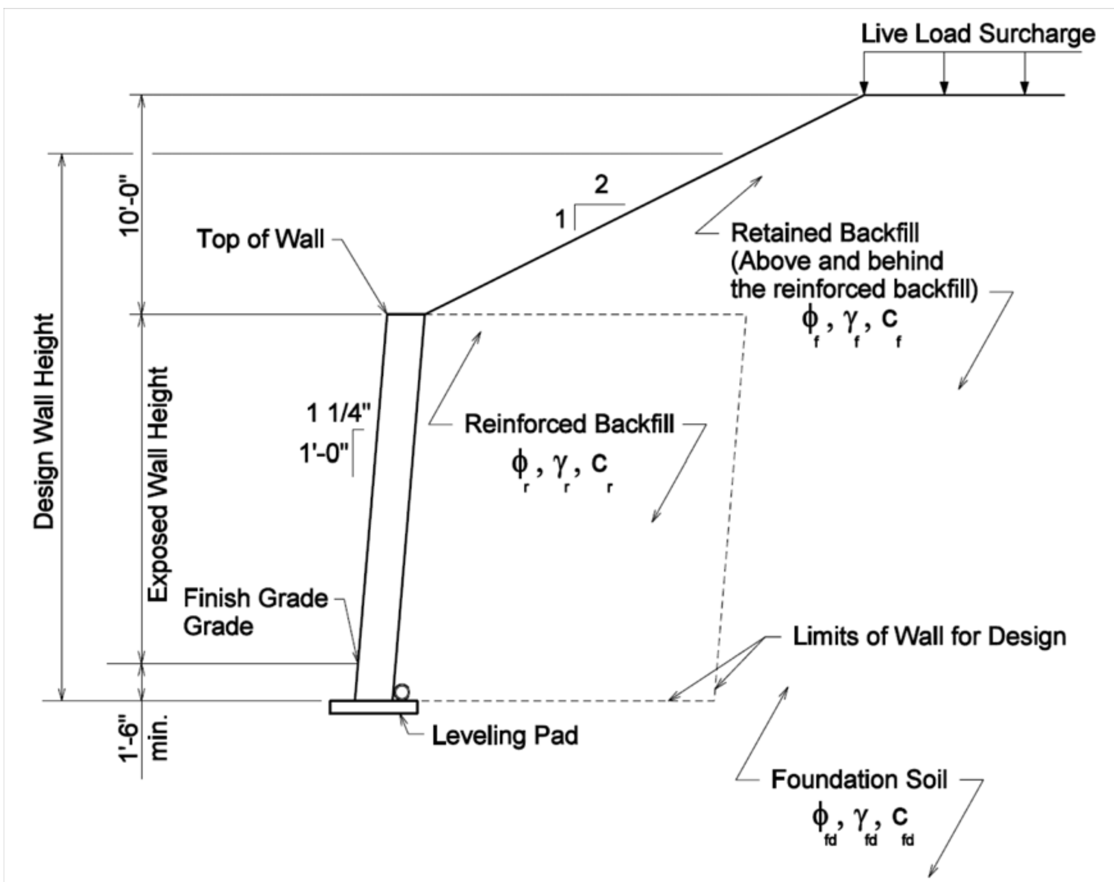

Figure E14-3.1-1
MSE Wall with Broken Backslope & Traffic

Wall Geometry

$H_e = 14.5$ Exposed wall height, ft

$H = H_e + 1.5$ Design wall height, ft (assume 1.5 ft wall embedment)

$\beta = 26.565\ deg$ Inclination of ground slope behind face of wall (2H:1V)

$b_1 = 1.25$ Front wall batter, in/ft (b_1H:12V)

$h_{slope} = 10.0$ Slope height, ft

$Batter = atan\left(\frac{b_1}{12}\right)$ Angle of front face of wall to vertical

$\boxed{Batter = 5.95\,deg}$

Note: Since the wall has less than 10 degrees of batter the wall can be defined as "near vertical" thus θ = 90 degrees and $\beta'=\delta'=I$ for a broken backslope

$\theta = 90\ deg$ Angle of back face of wall to horizontal

$I = atan\left(\frac{h_{slope}}{2\,H}\right)$ Infinite slope angle

$\boxed{I = 17.4\,deg}$

$\beta' = I$ Inclination of ground slope behind face of wall, deg

$\delta' = I$ Friction angle between fill and wall, deg

E14-3.2 Design Parameters

Project Parameters

$Design\_Life = 75$ Wall design life, years (min) **LRFD [11.5.1]**

Soil Properties (From Geotechnical Site Investigation Report)

Designer to determine if long-term or short-term soil strength parameters govern external stability.

Reinforced Backfill Soil Design Parameters

$\phi_r = 30\ deg$ Angle of internal friction **LRFD [11.10.5.1]** and (14.4.6)

$\gamma_r = 0.120$ Unit of weight, kcf

$c_r = 0$ Cohesion, psf

Retained Backfill Soil Design Parameters

$\phi_f = 29\ deg$ Angle of internal friction

$\gamma_f = 0.120$ Unit of weight, kcf

$c_f = 0$ Cohesion, psf

Foundation Soil Design Parameters

$\phi_{fd} = 31deg$ Angle of internal friction

$\gamma_{fd} = 0.125$ Unit of weight, kcf

$c_{fd} = 0$ Cohesion, psf

Factored Bearing Resistance of Foundation Soil

$q_R = 6.5$ Factored resistance at the strength limit state, ksf

Note: The factored bearings resistance, q_R, was assumed to be given in the Site Investigation Report. If not provided q_R shall be determined by calculating the nominal bearing resistance, q_n, per **LRFD [Eq 10.6.3.1.2a-1]** and factored with the bearing resistance factor,ϕ_b, for MSE walls (i.e., $q_R=\phi_b q_n$).

Precast Concrete Panel Facing Parameters

$S_v = 1.333$ Vertical spacing of reinforcement, ft

Note: vertical spacing should not exceed 2.7 ft without full scale test data **LRFD [11.10.6.2.1]**

Soil Reinforcement Design Parameters

Geosynthetic - Geogrids Reinforcing type

Note: Product specific information to be defined during internal stability checks

Live Load Surcharge Parameters

$h_{eq} = 2.0$ Equivalent height of soil for surcharge load, ft (14.4.5.4.2)

$SUR = h_{eq}\,\gamma_f$ Live load soil for surcharge load

$\boxed{SUR = 0.240}$ ksf/ft

Resistance Factors

$\phi_s = 1.00$ Sliding of MSE wall at foundation **LRFD [Table 11.5.6-1]**

$\phi_b = 0.65$ Bearing resistance **LRFD [Table 11.5.6-1]**

$\phi_t = 0.90$ Tensile resistance (geosynthetic reinforcement and connectors) **LRFD [Table 11.5.6-1]**

$\phi_p = 0.90$ Pullout resistance **LRFD [Table 11.5.6-1]**

E14-3.3 Estimate Depth of Embedment and Length of Reinforcement

For this example it is assumed that global stability does not govern the required length of soil reinforcement.

Embedment Depth, d_e

Frost-susceptible material is assumed to be not present or that it has been removed and replaced with nonfrost susceptible material per **LRFD [11.10.2.2]**. There is also no potential for scour. Therefore, the minimum embedment, d_e, shall be the greater of 1.5 ft (14.6.4) or H/20 **LRFD [Table C11.10.2.2-1]**

Note: While AASHTO allows the d_e value of 1.0 ft on level ground, the embedment depth is limited to 1.5 ft by WisDOT policy as stated in Chapter 14.

$$\frac{H}{20} = 0.8 \text{ ft}$$

$$d_e = \max\left(\frac{H}{20}, 1.5\right) \qquad \boxed{d_e = 1.50} \text{ ft}$$

Therefore, the initial design wall height assumption was correct.

$$H_e = 14.5 \text{ ft}$$

$$H = H_e + 1.5 \qquad \boxed{H = 16.00} \text{ ft}$$

Length of Reinforcement, L

In accordance with **LRFD [11.10.2.1]** the minimum required length of soil reinforcement shall be the greater of 8 feet or 0.7H. Due to the sloping backfill and traffic surcharge a longer reinforcement length of 0.9H will be used in this example. The length of reinforcement will be uniform throughout the entire wall height.

$$0.9\,H = 14.4 \text{ ft}$$

$$L_{user} = 14.5 \text{ ft}$$

$$L = \max(8.0, 0.9\,H, L_{user}) \qquad \boxed{L = 14.50} \text{ ft}$$

Height of retained fill at the back of the reinforced soil, h

$$h = H + L\tan(\beta) \qquad \boxed{h = 23.25} \text{ ft}$$

E14-3.4 Permanent and Transient Loads

In this example, load types EV (vertical earth pressure), EH (horizontal earth pressure) and LS (live load surcharge) will be used as shown in Figure E14-3.4-1. No transient loads are present in this example. Due to the relatively thin wall thickness the weight and width of the concrete facing will be ignored. Passive soil resistance will also be ignored.

E14-3.4.1 Compute Active Earth Pressure

Compute the coefficient of active earth pressure (k_a) using Coulomb Theory **LRFD [Eq 3.11.5.3-1]** with the wall backfill material interface friction angle, δ, set equal to β (i.e. δ=β) **LRFD [11.10.5.2].** The retained backfill soil will be used (i.e., k_a=k_{af})

$\phi_f = 29\,\text{deg}$

$\beta' = 17.4\,\text{deg}$

$\theta = 90\,\text{deg}$

$\delta' = 17.4\,\text{deg}$

$$\Gamma = \left(1 + \sqrt{\frac{\sin(\phi_f + \delta')\,\sin(\phi_f - \beta')}{\sin(\theta - \delta')\,\sin(\theta + \beta')}}\right)^2 \qquad \boxed{\Gamma = 1.961}$$

$$k_{af} = \frac{\sin(\theta + \phi_f)^2}{\Gamma\,\sin(\theta)^2\,\sin(\theta - \delta')} \qquad \boxed{k_{af} = 0.409}$$

E14-3.4.2 Compute Unfactored Loads

The forces and moments are computed using Figure E14-3.4-1 by their appropriate LRFD load types **LRFD [Tables 3.4.1-1 and 3.4.1-2]**

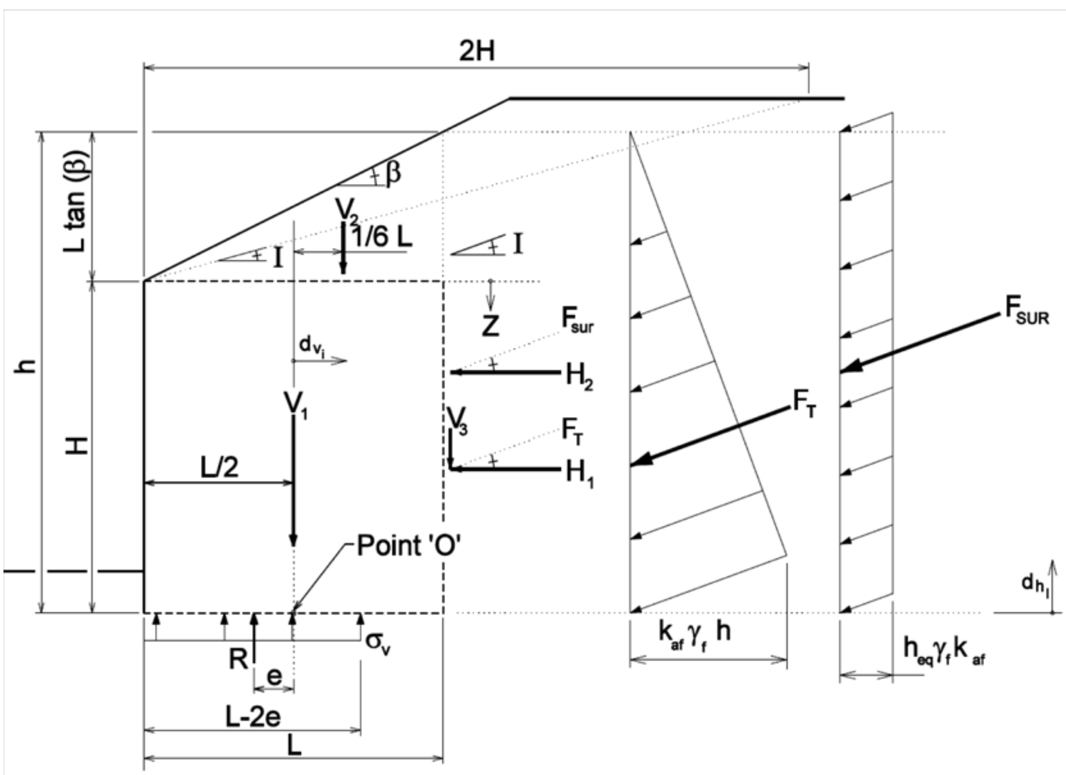

Figure E14-3.4-1
MSE Wall - External Stability

Active Earth Force Resultant, (kip/ft), F_T

$F_T = \frac{1}{2} \gamma_f h^2 k_{af}$ Active earth force resultant (EH) $\boxed{F_T = 13.3}$

Live Load Surcharge, (kip/ft), F_{SUR}

$F_{SUR} = h_{eq} \gamma_f h k_{af}$ Live load surcharge resultant (LS) $\boxed{F_{SUR} = 2.3}$

Vertical Loads, (kip/ft), V_i

$V_1 = \gamma_r H L$ Soil backfill - reinforced soil (EV) $\boxed{V_1 = 27.8}$

$V_2 = \frac{1}{2} \gamma_f L (L \tan(\beta))$ Soil backfill - backslope (EV) $\boxed{V_2 = 6.3}$

$V_3 = F_T \sin(I)$ Active earth force resultant (vertical component - EH) $\boxed{V_3 = 4}$

Moments produced from vertical loads about the center of reinforced soil, (kip-ft/ft) MV_i

Moment Arm		Moment	
$d_{v1} = 0$	$\boxed{d_{v1} = 0.0}$	$MV_1 = V_1 d_{v1}$	$\boxed{MV_1 = 0.0}$
$d_{v2} = \frac{1}{6} L$	$\boxed{d_{v2} = 2.4}$	$MV_2 = V_2 d_{v2}$	$\boxed{MV_2 = 15.2}$
$d_{v3} = \frac{L}{2}$	$\boxed{d_{v3} = 7.3}$	$MV_3 = V_3 d_{v3}$	$\boxed{MV_3 = 28.7}$

Horizontal Loads, (kip/ft), H_i

$H_1 = F_T \cos(I)$ Active earth force resultant (horizontal component - EH) $\boxed{H_1 = 12.7}$

$H_2 = F_{SUR} \cos(I)$ Live load surcharge resultant (LS) $\boxed{H_2 = 2.2}$

Moments produced from horizontal loads about the center of reinforced soil, (kip-ft/ft), MH_i

Moment Arm / Moment

$d_{h1} = \frac{h}{3}$ $\boxed{d_{h1} = 7.7}$ $MH_1 = H_1 d_{h1}$ $\boxed{MH_1 = 98.0}$

$d_{h2} = \frac{h}{2}$ $\boxed{d_{h2} = 11.6}$ $MH_2 = H_2 d_{h2}$ $\boxed{MH_2 = 25.3}$

Summary of Unfactored Forces & Moments:

Load			Moment Arm		Moment		LRFD Load Type
Item	Description	Value (kip/ft)	Item	Value (ft)	Item	Value (kip-ft/ft)	
V_1	Soil backfill	27.80	d_{v1}	0.0	MV_1	0.0	EV
V_2	Soil backfill	6.30	d_{v2}	2.4	MV_2	15.2	EV
V_3	Active earth pressure	4.00	d_{v3}	7.3	MV_3	28.7	EH

Table E14-3.4-1
Unfactored Vertical Forces & Moments

Load			Moment Arm		Moment		LRFD Load Type
Item	Description	Value (kip/ft)	Item	Value (ft)	Item	Value (kip-ft/ft)	
H_1	Active earth pressure	12.70	d_{h1}	7.7	MH_1	98.0	EH
H_2	Live Load Surcharge	2.20	d_{h2}	11.6	MH_2	25.3	LS

Table E14-3.4-2
Unfactored Horizontal Forces & Moments

E14-3.4.3 Summarize Applicable Load and Resistance Factors

Maximum and minimum load factors shall be used to determine the extreme load effects. WisDOT's policy is to set all load modifiers to one ($n = 1.0$). Factored loads and moments for each limit state are calculated by applying the appropriate load factors **LRFD [Tables 3.4.1-1 and 3.4.1-2]**. The following load combinations will be used in this example:

Load Combination Limit State	EV	LS	EH
Strength Ia (minimum)	$\gamma_{EVmin} = 1.00$	$\gamma_{LSmin} = 1.75$	$\gamma_{EHmin} = 0.90$
Strength Ib (maximum)	$\gamma_{EVmax} = 1.35$	$\gamma_{LSmax} = 1.75$	$\gamma_{EHmax} = 1.50$
Service I (max/min)	$\gamma_{EV} = 1.00$	$\gamma_{LS} = 1.00$	$\gamma_{EH} = 1.00$

Load Combination	γ_{EV}	γ_{LS_V}	γ_{LS_H}	γ_{EH}	Application
Strength Ia	1.00	-	1.75	1.50	Sliding, Eccentricity
Strength Ib	1.35	1.75	1.75	1.50	Bearing, T_{max}
Service I	1.00	1.00	1.00	1.00	Pullout (σ_v)

Table E14-3.4-3
Load Combinations

Load Combination Assumptions:

- Live load surcharge stabilizing loads (if applicable) are ignored for overturning and sliding analyses. Live load surcharge is used to compute maximum bearing pressure, wall strength and overall (global) stability.
- Minimum horizontal earth pressure ,$\gamma_{EH(min)}$ = 0.9, will not control in this example based on B/H and lateral load inclination, but should be checked.
- Component load factors shall remain consistent throughout calculations. For example, the active earth force resultant (F_T) can be broken into component forces of either $V_3\gamma_{EH(max)}$ and $H_1\gamma_{EH(max)}$ or $V_3\gamma_{EH(min)}$ and $H_1\gamma_{EH(min)}$, not $V_3\gamma_{EH(min)}$ and $H_1\gamma_{EH(max)}$.
- T_{max1} (Pullout) is calculated without live load and T_{max2} (Rupture) is calculated with live load.

The loads discussed and tabulated previously can now be factored by the appropriate load factors and combined to determine the governing limit states for each design check.

E14-3.4.4 Compute Factored Loads and Moments

Unfactored loads by load type (kip/ft)

$$V_{EV} = V_1 + V_2$$ $\boxed{V_{EV} = 34.1}$

$$V_{EH} = V_3$$ $\boxed{V_{EH} = 4.0}$

$$H_{EH} = H_1$$ $\boxed{H_{EH} = 12.7}$

$$H_{LS} = H_2$$ $\boxed{H_{LS} = 2.2}$

Unfactored moments by load type (kip-ft/ft)

$$M_{EV} = MV_1 + MV_2$$ $\boxed{M_{EV} = 15.2}$

$$M_{EH1} = MV_3$$ $\boxed{M_{EH1} = 28.7}$

$$M_{EH2} = MH_1$$ $\boxed{M_{EH2} = 98.0}$

$$M_{LS2} = MH_2$$ $\boxed{M_{LS2} = 25.3}$

Factored vertical loads by limit state (kip/ft)

$$V\_Ia = n\left(1.00V_{EV} + 1.50\ V_{EH}\right)$$ $\boxed{V\_Ia = 40.1}$

$$V\_Ib = n\left(1.35V_{EV} + 1.50\ V_{EH}\right)$$ $\boxed{V\_Ib = 52.0}$

$$V\_Ser = n\left(1.00V_{EV} + 1.00\ V_{EH}\right)$$ $\boxed{V\_Ser = 38.1}$

Factored horizontal loads by limit state (kip/ft)

$$H\_Ia = n\left(1.75H_{LS} + 1.50H_{EH}\right)$$ $\boxed{H\_Ia = 22.8}$

$$H\_Ib = n\left(1.75H_{LS} + 1.50H_{EH}\right)$$ $\boxed{H\_Ib = 22.8}$

$$H\_Ser = n\left(1.00H_{LS} + 1.00H_{EH}\right)$$ $\boxed{H\_Ser = 14.8}$

Factored moments produced by vertical Loads by limit state (kip-ft/ft)

$$MV\_Ia = n\left(1.00M_{EV} + 1.50\ M_{EH1}\right)$$ $\boxed{MV\_Ia = 58.2}$

$$MV\_Ib = n\left(1.35M_{EV} + 1.50\ M_{EH1}\right)$$ $\boxed{MV\_Ib = 63.6}$

$$MV\_Ser = n\left(1.00M_{EV} + 1.00\ M_{EH1}\right)$$ $\boxed{MV\_Ser = 43.9}$

Factored moments produced by horizontal loads by limit state (kip-ft/ft)

$$MH\_Ia = n\left(1.75M_{LS2} + 1.50\ M_{EH2}\right)$$ $\boxed{MH\_Ia = 191.3}$

$$MH\_Ib = n\left(1.75M_{LS2} + 1.50\ M_{EH2}\right)$$ $\boxed{MH\_Ib = 191.3}$

$$MH\_Ser = n\left(1.00M_{LS2} + 1.00\ M_{EH2}\right)$$ $\boxed{MH\_Ser = 123.3}$

Summary of Factored Forces & Moments:

Load Combination	Vert. Loads V (kips/ft)	Moments MV (kip-kip/ft)	Horiz. Loads H (kips/ft)	Moments MH (kip-kip/ft)
Strength Ia	40.1	58.2	22.8	191.3
Strength Ib	52.0	63.6	22.8	191.3
Service I	38.1	43.9	14.8	123.3

Table E14-3.4-4
Summary of Factored Loads & Moments

E14-3.5 Evaluate External Stability of MSE Wall

Three potential external failure mechanisms will be considered in this example (sliding at the base, limiting eccentricity and bearing resistance). Overall (global) stability requirements are not included here. Design calculations will be carried out for the governing limit states only.

E14-3.5.1 Sliding Resistance at Base of MSE Wall

The following calculations are based on **Strength Ia**:

Factored Sliding Force

$R_u = H\_Ia$ $\boxed{R_u = 22.8}$ kip/ft

Sliding Resistance

To compute the coefficient of sliding friction for continuous reinforcement use the lesser friction angle of the reinforced back fill, ϕ_r , or the foundation soil, ϕ_{fd}, **LRFD [11.10.5.3]**.

$\phi_\mu = \min(\phi_r, \phi_{fd})$ $\boxed{\phi_\mu = 30\,deg}$

Note: Since continuous reinforcement is used, a slip plane may occur at the reinforcement layer. The sliding friction angle for this case shall use the lesser of (when applicable) ϕ_r, ϕ_{fd}, and ρ. Where ρ is the soil-reinforcement interface friction angle. Without specific data ρ may equal 2/3 ϕ_f with ϕ_f a maximum of 30 degrees. This check is not made in this example, but is required.

$\mu = \tan(\phi_\mu)$ $\boxed{\mu = 0.577}$

$V\_Ia = 40.1$ Factored vertical load, kip/ft

$V_{Nm} = \mu\ V\_Ia$ $\boxed{V_{Nm} = 23.1}$ kip/ft

$\phi_s = 1.00$

$R_R = \phi_s V_{Nm}$ $\boxed{R_R = 23.1}$ kip/ft

Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} = \frac{R_R}{R_u}$$ $$\boxed{CDR_{Sliding} = 1.02}$$

Is the CDR $\geq$ 1.0 ? $\boxed{check = "OK"}$

E14-3.5.2 Limiting Eccentricity at Base of MSE Wall

The location of the resultant of the reaction forces is limited to the middle one-half of the base width for a soil foundation (i.e., e_{max} = L/4) **LRFD [11.6.3.3]**. The following calculations are based on **Strength Ia**.

Maximum eccentricity

$$e_{max} = \frac{L}{4}$$ $$\boxed{e_{max} = 3.63} \text{ ft}$$

Compute wall eccentricity (distance from Point 'O' Figure E14-3.4-1)

$\Sigma M_R = MV\_Ia$ Summation of resisting moments for Strength Ia

$\Sigma M_O = MH\_Ia$ Summation of overturning moments for Strength Ia

$\Sigma V = V\_Ia$ Summation of vertical loads for Strength Ia

$\boxed{\Sigma M_R = 58.2}$ kip-ft/ft

$\boxed{\Sigma M_O = 191.3}$ kip-ft/ft

$\boxed{\Sigma V = 40.1}$ kip/ft

$$e = \frac{\Sigma M_O - \Sigma M_R}{\Sigma V}$$ $$\boxed{e = 3.32} \text{ ft}$$

Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} = \frac{e_{max}}{e}$$ $$\boxed{CDR_{Eccentricity} = 1.09}$$

Is the CDR $\geq$ 1.0 ? $\boxed{check = "OK"}$

E14-3.5.3 Bearing Resistance at base of MSE Wall

The following calculations are based on **Strength Ib**:

Compute wall eccentricity (distance from Point 'O' Figure E14-3.4-1)

$\Sigma M_R = MV\_Ib$ Summation of resisting moments for Strength Ib

$\Sigma M_O = MH\_Ib$ Summation of overturning moments for Strength Ib

$\Sigma V = V\_Ib$ Summation of vertical loads for Strength Ib

$\boxed{\Sigma M_R = 63.6}$ kip-ft/ft

$\boxed{\Sigma M_O = 191.3}$ kip-ft/ft

$\boxed{\Sigma V = 52.0}$ kip/ft

$$e = \frac{\Sigma M_O - \Sigma M_R}{\Sigma V}$$

$\boxed{e = 2.46}$ ft

Compute the ultimate bearing stress

σ_V = Ultimate bearing stress
L = Bearing length
e = Eccentricity (resultant produced by extreme bearing resistance loading)

Note: For the bearing resistance calculations the effective bearing width, B' = L-2e, is used instead of the actual width. Also, when the eccentricity, e, is negative: B'=L. The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**.

$$\sigma_V = \frac{\Sigma V}{L - 2\,e}$$

$\boxed{\sigma_V = 5.43}$ ksf/ft

Factored bearing resistance

$\boxed{q_R = 6.50}$ ksf/ft

Capacity:Demand Ratio (CDR)

$$CDR_{Bearing} = \frac{q_R}{\sigma_V}$$

$\boxed{CDR_{Bearing} = 1.20}$

Is the $CDR \geq 1.0$?

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

E14-3.6 Evaluate Internal Stability of MSE Wall

Note: MSE walls are a proprietary wall system and the internal stability computations shall be performed by the wall supplier.

Internal stability shall be checked for 1) pullout and 2) rupture in accordance with **LRFD [11.10.6]**. The factored tensile load, T_{max}, is calculated twice for internal stability checks for vertical stress (σ_V) calculations. For pullout T_{max1} is determined by excluding live load surcharge. For rupture T_{max2} is determined by including live load surcharge. In this example, the maximum reinforcement loads are calculated using the Simplified Method.

The location of the potential failure surface for a MSE wall with metallic strip or grid reinforcements (inextensible) is shown in Figure E14-2.6-1.

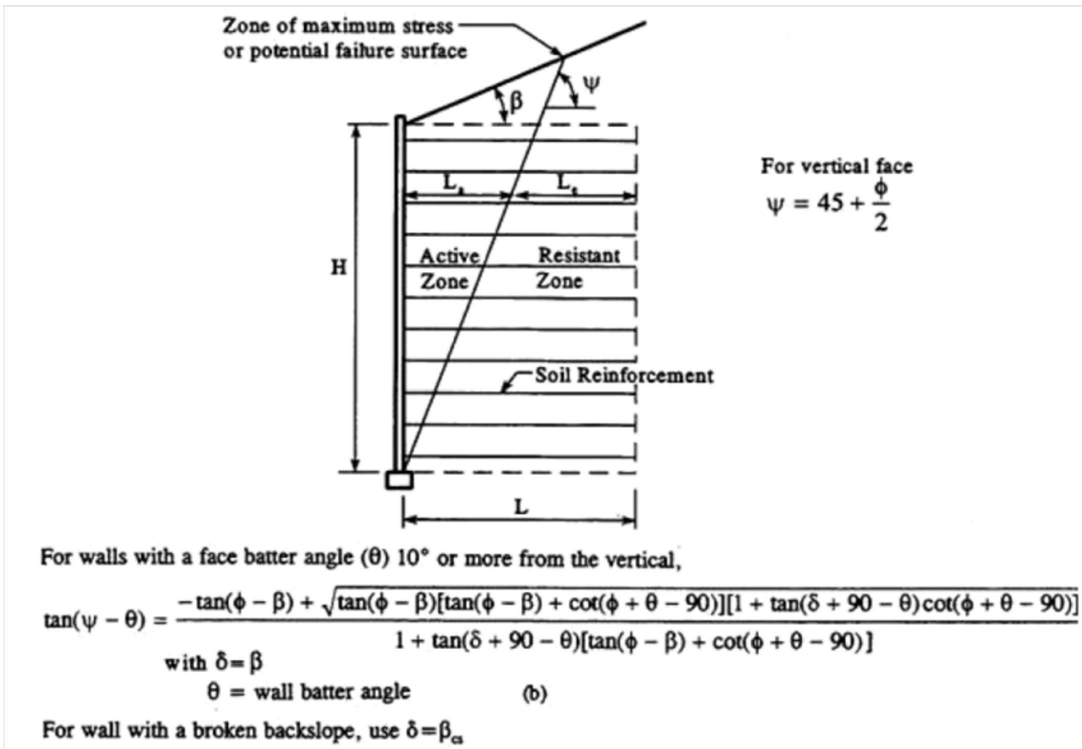

Figure E14-2.6-1
MSE Wall - Internal Stability (Extensible Reinforcement)
FHWA [Figure 4-9]

E14-3.6.1 Establish the Vertical Layout of Soil Reinforcement

Soil reinforcement layouts are shown in Table E14-3.6-1. They were determined by a standard block wall unit thickness of 8-in and a maximum vertical reinforcement spacing of 2.7-ft. The top and bottom level vertical spacing was adjusted to fit the height of the wall. Computations for determining the maximum tension, T_{max}, are taken at each level in the vertical layout.

$Layer = 3$ Layer of reinforcement (from top)

$Z = 3.333$ ft Depth below top of wall, ft

$S_V = 1.33$ ft Vertical spacing of reinforcement, ft

Calculate the upper and lower tributary depths based on the reinforcement vertical spacing

$$Z_{neg} = Z - \frac{S_V}{2}$$ $\boxed{Z_{neg} = 2.67}$ ft

$$Z_{pos} = Z + \frac{S_V}{2}$$ $\boxed{Z_{pos} = 4.00}$ ft

Layer	Z (ft)	Zneg (ft)	Zpos (ft)
1	0.67	0.00	1.33
2	2.00	1.33	2.67
3	3.33	2.67	4.00
4	4.67	4.00	5.33
5	6.00	5.33	6.67
6	7.33	6.67	8.00
7	8.67	8.00	9.33
8	10.00	9.33	10.67
9	11.33	10.67	12.00
10	12.67	12.00	13.33
11	14.00	13.33	14.67
12	15.33	14.67	16.00

Table E14-3.6-1
Vertical Layout of Soil Reinforcement

E14-3.6.2 Compute Horizontal Stress and Maximum Tension, T_{max}

Factored horizontal stress

$$\sigma_H = \gamma_P\,(\sigma_v k_r + \Delta\sigma_H)$$ **LRFD [Eq 11.10.6.2.1-1]**

γ_p = Load factor for vertical earth pressure (γ_{EVmax})

k_r = Horizontal pressure coefficient

σ_v = Pressure due to gravity and surcharge for pullout, T_{max1} ($\gamma_r Z_{trib} + \sigma_2$)

σ_v = Pressure due to gravity and surcharge for pullout resistance ($\gamma_r Z_{p\text{-}PO}$)

σ_v = Pressure due to gravity and surcharge for rupture, T_{max2} ($\gamma_r Z_{trib} + \sigma_2 + q$)

$\Delta\sigma_H$ = Horizontal pressure due to concentrated horizontal surcharge load

Z = Reinforcement depth for max stress Figure E14-2.6-2

Z_p = Depth of soil at reinforcement layer potential failure plane

$Z_{p\text{-}ave}$ = Average depth of soil at reinforcement layer in the effective zone

σ_2 = Equivalent uniform stress from backslope $(0.5(0.7)L\tan\beta)\gamma_f$

q = Surcharge load ($q = SUR$), ksf

To compute the lateral earth pressure coefficient, k_r , a k_a multiplier is used to determine k_r for each of the respective vertical tributary spacing depths (Z_{pos}, Z_{neg}). The k_a multiplier is determined using Figure E14-2.6-2. To calculate k_a it is assumed that $\delta=\beta$ and $\beta = 0$; thus, $k_a=\tan^2(45-\phi_f/2)$ **LRFD [Eq C11.10.6.2.1-1]**

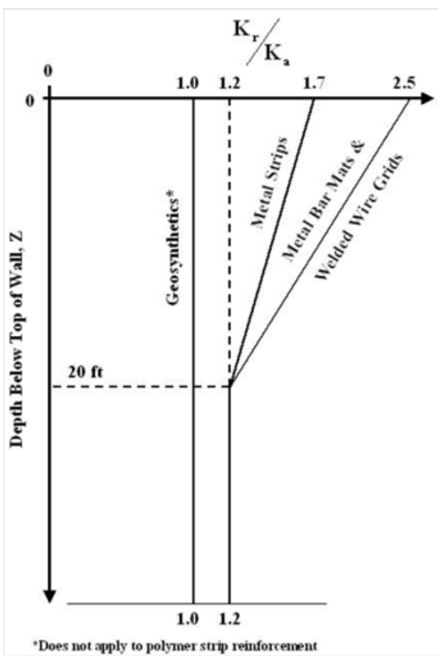

Figure E14-3.6-2
k_r/k_a Variation with MSE Wall Depth
FHWA [Figure 4-10]

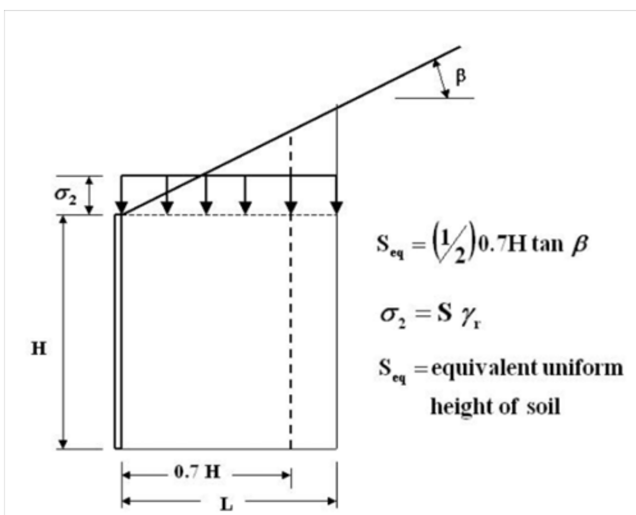

Figure E14-3.6-3
Calculation of Vertical Stress
FHWA [Figure 4-11]

Calculate the coefficient of active earth pressure, k_a

$$\phi_r = 30\,deg$$

$$k_a = \tan\left(45\ deg - \frac{\phi_r}{2}\right)^2 \qquad \boxed{k_a = 0.333}$$

The internal lateral earth pressure coefficient, k_r, for geogrids remains constant throughout the reinforced soil zone. k_r will be equal to $k_a(k_r/k_a) = k_a$ at any depth below the top of wall as shown in figure E14-3.6-2 **LRFD [Figure 11.10.6.2.2-3]**.

$$k_r = k_a \qquad \boxed{k_r = 0.333}$$

Compute effective (resisting) length, L_e

$$Z = 3.33 \quad ft$$

$$H = 16.00 \quad ft$$

$$L = 14.5 \quad ft$$

$$\psi = 45\ deg + \frac{\phi_r}{2} \qquad \boxed{\psi = 60.0\,deg}$$

$$L_a = \frac{H - Z}{\tan(\psi)} \qquad \boxed{L_a = 7.31}$$

$$L_e = \max(L - L_a, 3) \qquad \boxed{L_e = 7.19}$$

Note: L_e shall be greater than or equal to 3 ft **LRFD [11.10.6.3.2]**

E14-3.6.3 Establish Nominal and Factored Pullout Resistance of Soil Reinforcement

Compute the factored horizontal stress, σ_H, at Z by averaging the upper and lower tributary values (Z_{neg} and Z_{pos}). Since there is no horizontal stresses from concentrated dead loads values $\Delta\sigma_H$ is set to zero.

$$\sigma_H = \gamma_{EVmax} (\gamma_r Z_{trib} + \sigma_2) k_r$$

Surcharge loads

$$\sigma_2 = \frac{1}{2} 0.7\ H \tan(\beta)\ \gamma_f \qquad \boxed{\sigma_2 = 0.336}\ \text{ksf/ft}$$

Horizontal stress at Z_{neg} and Z_{pos}

$$\sigma_{H\_neg} = \gamma_{EVmax} (\gamma_r Z_{neg} + \sigma_2)\ k_r \qquad \boxed{\sigma_{H\_neg} = 0.295}\ \text{ksf/ft}$$

$$\sigma_{H\_pos} = \gamma_{EVmax} (\gamma_r Z_{pos} + \sigma_2)\ k_r \qquad \boxed{\sigma_{H\_pos} = 0.367}\ \text{ksf/ft}$$

Horizontal stress at Z

$$\sigma_H = 0.5(\sigma_{H\_pos} + \sigma_{H\_neg}) \qquad \boxed{\sigma_H = 0.331}\ \text{ksf/ft}$$

Compute the maximum tension, $T_{max\ 1}$, at Z

$$S_v = 1.33 \quad \text{ft}$$

$$T_{max1} = \sigma_H S_v 1000. \qquad \boxed{T_{max1} = 441}\ \text{plf}$$

Compute effective vertical stress for pullout resistance, σ_v

$$Z_{p\_PO} = Z + 0.5 \tan(\beta) (L_a + L) \qquad \boxed{Z_{p\_PO} = 8.8}\ \text{ft}$$

$\gamma_{EV} = 1.00$ Unfactored vertical stress for pullout resistance **LRFD [11.10.6.3.2]**

$$\sigma_v = \gamma_{EV} \gamma_r Z_{p\_PO} 1000 \qquad \boxed{\sigma_v = 1054}\ \text{psf}$$

Compute pullout resistance factor, F*

Pullout resistance factor, F*, for extensible geosynthetic reinforcement remains constant throughout the reinforced soil for determining the internal lateral earth pressure. Since no product-specific pullout test data is provided at the time of design F* and the scale effect correction factor,α, default values will be used per **LRFD [Figure 11.10.6.3.2-1 and Table 11.10.6.3.2-1]**.

Use default values for F' and α since product-specific pullout test data has not been provided.

$F' = 0.67 \tan(\phi_r)$ Pullout Friction Factor (Geogrids F*= 0.67tanϕ'_r Default value) **LRFD [Figure 11.10.6.3.2-1]**

$$\boxed{F' = 0.387}$$

Compute nominal pullout resistance, P_r

$\alpha = 0.8$	Scale effect correction factor (geogrids α = 0.8 default value) **LRFD [Table 11.10.6.3.2-1]**
$C = 2$	Overall reinforcement surface area geometry factor (geogrids C = 2.0) **LRFD [11.10.6.3.2]**
$R_c = 1$	Reinforcement coverage ratio (continuous reinforcement R_c = 1.0) **LRFD [11.10.6.4]**

$P_r = F' \alpha \sigma_v C R_c L_e$ $\boxed{P_r = 4690}$ plf

Compute factored pullout resistance, P_{rr}

$\phi_p = 0.9$

$P_{rr} = \phi_p P_r$ $\boxed{P_{rr} = 4221}$ plf

E14-3.6.4 Establish Nominal and Factored Long-term Tensile Resistance of Soil Reinforcement

Compute the factored horizontal stress, σ_H

$$\sigma_H = \gamma_{EVmax} (\gamma_r Z + \sigma_2 + q) k_r$$

Surcharge loads

$\boxed{\sigma_2 = 0.34}$ ksf/ft

$\boxed{q = 0.24}$ ksf/ft

Horizontal stress at Z_{neg} and Z_{pos}

$\sigma_{H\_neg} = \gamma_{EVmax} \left(\gamma_r Z_{neg} + \sigma_2 + q\right) k_r$ $\boxed{\sigma_{H\_neg} = 0.40}$ ksf/ft

$\sigma_{H\_pos} = \gamma_{EVmax} \left(\gamma_r Z_{pos} + \sigma_2 + q\right) k_r$ $\boxed{\sigma_{H\_pos} = 0.48}$ ksf/ft

Horizontal stress at Z

$\sigma_H = 0.5\left(\sigma_{H\_pos} + \sigma_{H\_neg}\right)$ $\boxed{\sigma_H = 0.44}$ ksf/ft

Compute the maximum tension, T_{max2}, at Z

$S_v = 1.33$ ft

$T_{max2} = \sigma_H S_v 1000$ $\boxed{T_{max2} = 585}$ plf

$$T_r = \phi T_{al} = \phi T_{ult}/RF$$

T_r = Factored soil reinforcement tensile resistance
ϕ = Resistance factor
T_{al} = Nominal geosynthetic reinforcement strength
T_{ult} = Ultimate tensile strength
RF_{CR} = Creep reduction factor
RF_D = Durability reduction factor
RF_{ID} = Installation damage reduction factor
RF = Reduction factor (RF_{CR} x RF_D x RF_{ID})

The following calculation for determining the nominal long-term reinforcement tensile strength uses values similar to proprietary product specific data. In any application RF_{ID} nor RF_D shall not be less than 1.1. A single default reduction factor, RF, of 7 may be used for permanent applications if meeting the requirements listed in **LRFD [11.10.6.4.2b and Table 11.10.6.4.2b-1,Table 11.10.6.4.2b-1]**

	Geogrid Type		
	#1	#2	#3
T_{ult} (plf)	2500	5000	7500
RF_{CR}	2.00	2.00	2.00
RF_D	1.15	1.15	1.15
RF_{ID}	1.35	1.35	1.35

Table E14-3.6-2
Geogrid Resistance Properties

$\text{Grade} = 1$

$T_{ult} = 2500$ plf

$RF_{CR} = 2.00$

$RF_D = 1.15$

$RF_{ID} = 1.35$

$RF = RF_{CR}\, RF_D\, RF_{ID}$ $\qquad \boxed{RF = 3.11}$

$T_{al} = \dfrac{T_{ult}}{RF}$ $\qquad \boxed{T_{al} = 805}$ plf

$T_r = \phi_t\, T_{al}$ $\qquad \boxed{T_r = 725}$ plf

E14-3.6.5 Establish Grade of Soil Reinforcing Elements at Each Level

Based on Pullout Resistance

$$CDR_{pullout} = \frac{P_{rr}}{T_{max1}}$$ $\boxed{CDR_{pullout} = 9.56}$

Is the $CDR \geq 1.0$? $\boxed{check = "OK"}$

Based on Tensile Resistance

$$CDR_{tensile} = \frac{T_r}{T_{max2}}$$ $\boxed{CDR_{tensile} = 1.24}$

Is the $CDR \geq 1.0$? $\boxed{check = "OK"}$

Note: If the wall requires additional reinforcement the vertical spacing will be maintained and adjustments will be made to the grade (strength) for each layer accordingly.

E14-3.7 Summary of Results

E14-3.7.1 Summary of External Stability

Based on the defined project parameters, embedment depth and length of reinforcement the following external stability checks have been satisfied:

External Check	CDR
Sliding	1.02
Eccentricity	1.09
Bearing	1.20

Table E14-3.7-1
Summary of External Stability Computations

E14-3.7.2 Summary of Internal Stability

Computations for the grades of geogrid reinforcements at each level is presented in Table E14-3.7-2.

		Pullout			Rupture					
Level	Z	σ_H	T_{max1}	P_{rr}	Grade	σ_H	T_{max2}	T_r	CDR_p	CDR_t
1	0.67	187	250	2455	#1	295	394	725	9.84	1.84
2	2.00	259	346	3280	#1	367	490	725	9.49	1.48
3	3.33	331	442	4221	#1	439	586	725	9.56	1.24
4	4.67	403	538	5280	#1	511	682	725	9.82	1.06
5	6.00	475	634	6456	#2	583	778	1449	10.19	1.86
6	7.33	547	730	7750	#2	655	874	1449	10.62	1.66
7	8.67	619	826	9161	#2	727	970	1449	11.10	1.49
8	10.00	691	922	10690	#2	799	1066	1449	11.60	1.36
9	11.33	763	1018	12336	#2	871	1162	1449	12.12	1.25
10	12.67	835	1114	14099	#2	943	1258	1449	12.66	1.15
11	14.00	907	1210	15980	#2	1015	1354	1449	13.21	1.07
12	15.33	979	1306	17978	#3	1087	1450	2174	13.77	1.50

Table E14-3.7.2
Summary of Internal Stability Computations for Strength I Load Combinations

E14-3.8 Final MSE Wall Schematic

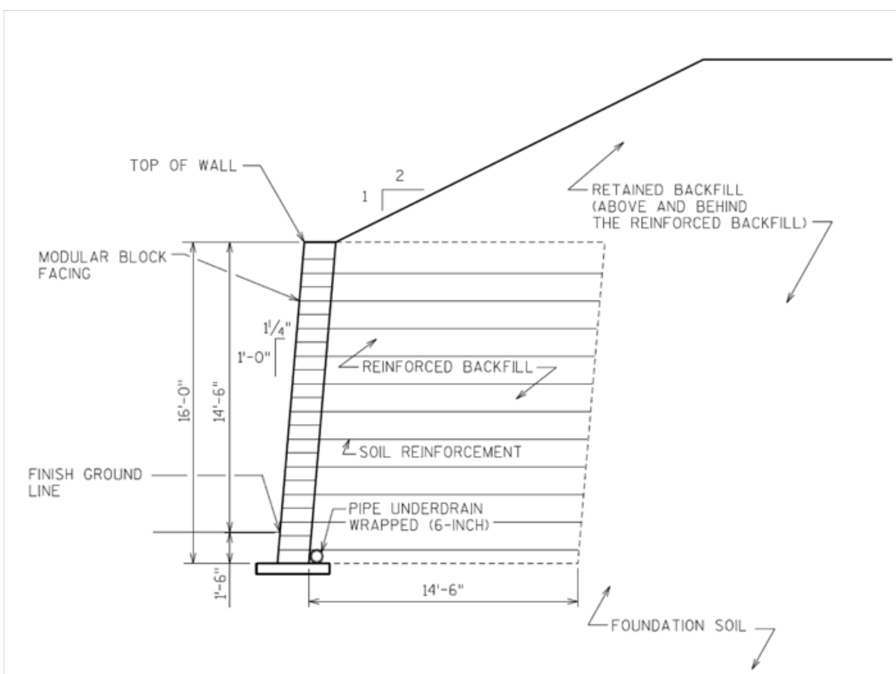

Figure E14-3.8-1
MSE Wall Schematic

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E14-4 Cast-In-Place Concrete Cantilever Wall on Piles, LRFD

General

This example shows design calculations for a cast-in-place (CIP) concrete wall supported on piles conforming to the LRFD Bridge Design Specifications and the WisDOT Bridge Manual. *(Example is current through LRFD Fifth Edition - 2010)*

Sample design calculations for pile capacities and wall strength design will be presented. The overall stability and settlement calculations will not be shown in this example, but are required.

Design steps presented in 14.5.2.1 are used for the wall design.

E14-4.1 Establish Project Requirements

The CIP concrete wall shown in Figure E14-6.1-1 will be designed appropriately to accommodate a horizontal backslope. External stability, overall stability and wall strength are the designer's (WisDOT/Consultant) responsibility.

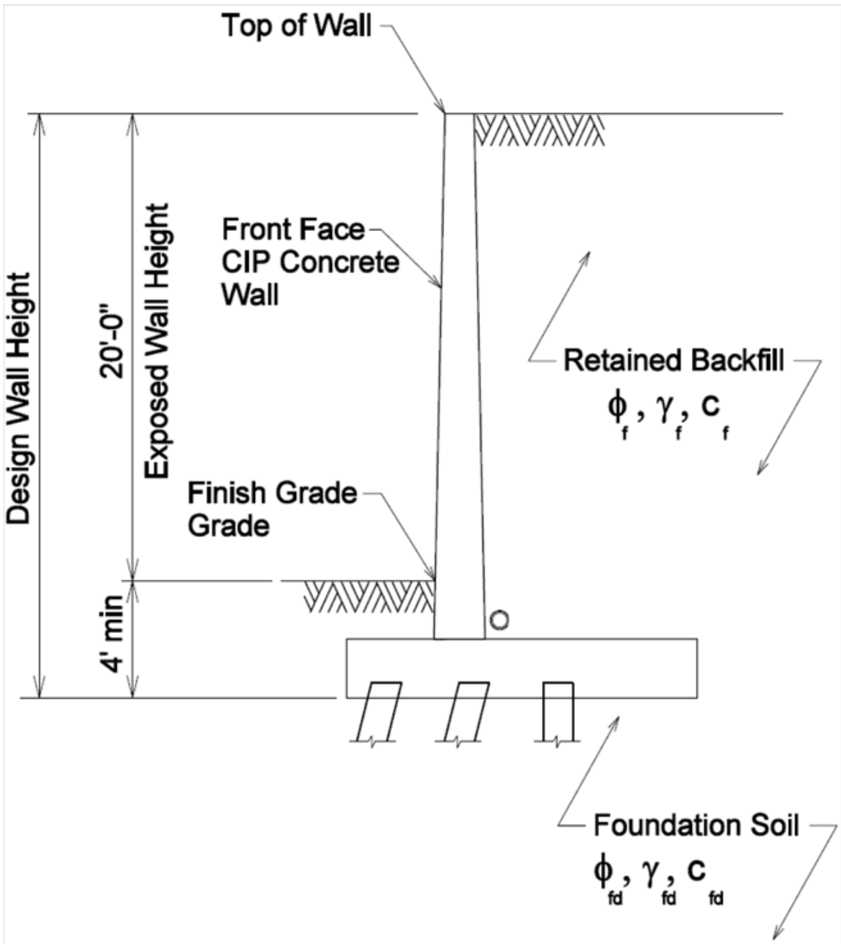


Figure E14-4.1-1
CIP Concrete Wall on Piles

E14-4.2 Design Parameters

Project Parameters

$Design\_Life = 75\ years$ Wall design life (min) **LRFD [11.5.1]**

Soil Properties (From Geotechnical Site Investigation Report)

Designer to determine if long-term or short-term soil strength parameters govern external stability.

Backfill Soil Design Parameters

$\phi_f = 32\ deg$ Angle of internal friction

$\gamma_f = 0.120$ Unit weight, kcf

$c_f = 0$ Cohesion, pcf

$\delta = 17\ deg$ Friction angle between fill and wall

Note: Per WisDOT Bridge Manual and Standard Specifications, structural backfill shall be granular and non-expansive.

ϕ_f= 32 degrees is used for this example, however ϕ_f=30 degrees is the maximum that should be used without testing.

Foundation Soil Design Parameters

$\phi_{fd} = 29\ deg$ Angle of internal friction

$\gamma_{fd} = 0.110$ Unit of weight, kcf

$c_{fd} = 0$ Cohesion, psf

Reinforced Concrete Parameters

$f'_c = 3.5$ Concrete compressive design strength, ksi (14.5.9)

$\gamma_c = 0.150$ $w_c = \gamma_c$ Unit weight of concrete, ksf

$E_c = 33000\ w_c^{1.5}\sqrt{f'_c}$ Modulus of elasticity of concrete, ksi **LRFD [5.4.2.4]**

$\boxed{E_c = 3587}$ ksi

$f_y = 60$ Yield strength of reinforcing bars, ksi (14.5.9)

$E_s = 29000$ Modulus of elasticity of reinforcing bars, ksi

Live Load Surcharge Parameters

Live load surcharge shall be used when vehicular load is located within H/2 of the backface of the wall **LRFD [3.11.6.4].** The equivalent height of soil for vehicular load, H_{eq}, used for surcharge loads shall be in accordance to **LRFD [Table 3.11.6.4-2]**. However, WisDOT policy for most cases requires an equivalent height of 2.0 feet. The following procedure is used for determining live load surcharge:

$L_{traffic} = 100.00$ Distance from wall backface to edge of traffic, ft

$\frac{H}{2} = 12.00$ Distance from wall backface where live load surcharge shall be considered in the wall design, ft

Note: The wall height used is the exposed height plus an assumed 4 feet embedment ($H=H_e+4$ feet).

Shall live load surcharge be included? check = "NO"

$h_{eq} = 0.833$ Equivalent height of soil for surcharge load, ft (14.4.5.4.2)

WisDOT Policy: Wall with live load from traffic use 2.0 feet (240 psf) and walls without traffic use 0.833 feet (100 psf)

E14-4.3 Define Wall Geometry

Wall Geometry

$H_e = 20.00$ Exposed wall height, ft

$D_f = 4.00$ Footing cover, ft (WisDOT policy 4'-0" minimum)

$H = H_e + D_f$ Design wall height, ft

$T_t = 1.00$ Stem thickness at top of wall, ft

$b_1 = 0.25$ Front wall batter, in/ft (b_1H:12V)

$b_2 = 0.50$ Back wall batter, in/ft (b_2H:12V)

$\beta = 0.00\,deg$ Inclination of ground slope behind face of wall, deg (horizontal)

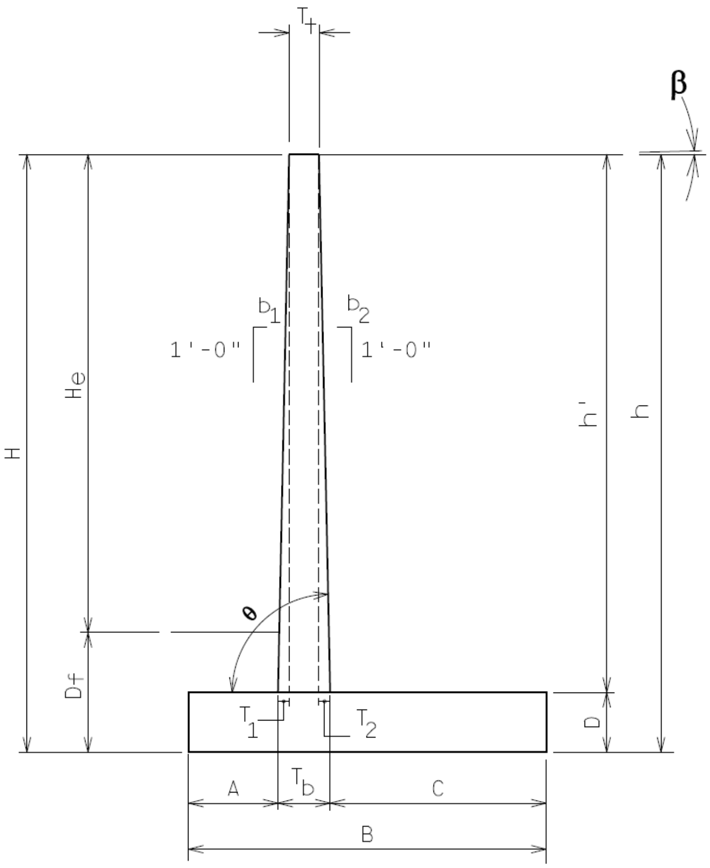


Figure E14-4.3-1
CIP Concrete Wall Geometry

Preliminary Wall Dimensioning

Selecting the most optimal wall configuration is an iterative process and depends on site conditions, cost considerations, wall geometry and aesthetics. For this example, the iterative process has been completed and the final wall dimensions are used for design checks.

$H = 24.0$	Design wall height, ft
$B = 12.00$	Footing base width, ft (2/5H to 3/5H)
$A = 4.75$	Toe projection, ft (H/8 to H/5)
$D = 2.50$	Footing thickness, ft (H/8 to H/5)

WisDOT policy: H≤10'-0" D_{min}=1'-6"
H>10'-0" D_{min}=2'-0"
On Piles D_{min}=2'-0"

Other Wall Dimensioning

$h' = H - D$	Stem height, ft	$h' = 21.5$
$T_1 = b_1 \frac{h'}{12}$	Stem front batter width, ft	$T_1 = 0.448$
$T_2 = b_2 \frac{h'}{12}$	Stem back batter width, ft	$T_2 = 0.896$
$T_b = T_1 + T_t + T_2$	Stem thickness at bottom of wall, ft	$T_b = 2.34$
$C = B - A - T_b$	Heel projection, ft	$C = 4.91$
$\theta = \operatorname{atan}\left(\frac{12}{b_2}\right)$	Angle of back face of wall to horizontal	$\theta = 87.6\,deg$
$b = 12$	Concrete strip width for design, in	
$h = H + (T_2 + C)\tan(\beta)$	Retained soil height, ft	$h = 24.0$

Pile Dimensioning

$y_{p1} = 1.25$	Distance from Point 'O' to centerline pile row 1, ft
$PS1 = 2.75$	Distance from centerline pile row 1 to centerline pile row 2, ft
$PS2 = 3.00$	Distance from centerline pile row 2 to centerline pile row 3, ft
$P_1 = 8.00$	Spacing between piles in row 1, ft
$P_2 = 8.00$	Spacing between piles in row 2, ft
$P_3 = 8.00$	Spacing between piles in row 3, ft

Pile Parameters (From Geotechnical Site Investigation Report, assuming HP12x53)

$Pile\_Axial = 220$	Pile axial capacity (factored), kips
$pile\_batter = 4$	Pile batter (pile_batterV:1H)
$H_{r1} = 11$	Pile row 1 lateral capacity (factored), kips*
$H_{r2} = 11$	Pile row 2 lateral capacity (factored), kips*
$H_{r3} = 14$	Pile row 3 lateral capacity (factored), kips*
$B_{xx} = 12.05$	Pile flange width (normal to wall alignment) dimension, in
$B_{yy} = 11.78$	Pile depth (perpendicular to wall alignment) dimension, in

* Based on LPILE or Broms' Method ϕ=1.0

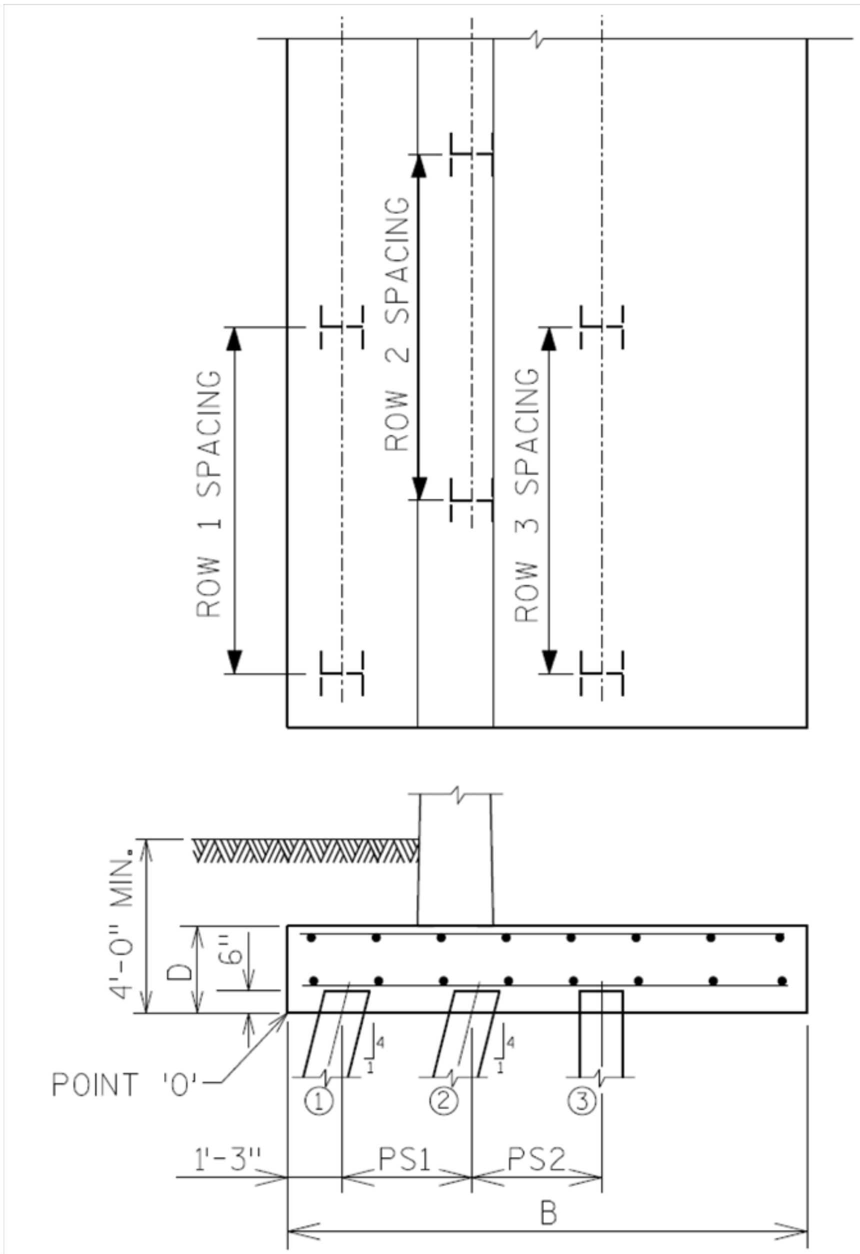


Figure E14-4.3-2
CIP Concrete Pile Geometry

E14-4.4 Permanent and Transient Loads

In this example, load types DC (dead load components), EV (vertical earth pressure), EH (horizontal earth pressure) and LS (live load surcharge) will be used. Passive resistance of the footing will be ignored.

E14-4.4.1 Compute Active Earth Pressure Coefficient

Compute the coefficient of active earth pressure using Coulomb Theory

LRFD [Eq 3.11.5.3-1]

$\phi_f = 32.0\,deg$

$\beta = 0.0\,deg$

$\theta = 87.6\,deg$

$\delta = 17.0\,deg$

$$k_a = \frac{\sin(\theta + \phi_f)^2}{\Gamma \sin(\theta)^2 \sin(\theta - \delta)}$$

$$\Gamma = \left(1 + \sqrt{\frac{\sin(\phi_f + \delta)\sin(\phi_f - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right)^2 \qquad \boxed{\Gamma = 2.727}$$

$$k_a = \frac{\sin(\theta + \phi_f)^2}{\Gamma \sin(\theta)^2 \sin(\theta - \delta)} \qquad \boxed{k_a = 0.294}$$

E14-4.4.2 Compute Pile Group Properties

Compute the distance from Point 'O' to the pile row centerlines

$$y_{p1} = 1.25 \qquad \boxed{y_{p1} = 1.25} \text{ ft}$$

$$y_{p2} = y_{p1} + PS1 \qquad \boxed{y_{p2} = 4.00} \text{ ft}$$

$$y_{p3} = y_{p1} + PS1 + PS2 \qquad \boxed{y_{p3} = 7.00} \text{ ft}$$

Compute the effective number of piles in each pile row and overall

$$NP_1 = \begin{cases} \dfrac{1}{P_1} & \text{if } P_1 > 0 \\ 0 & \text{otherwise} \end{cases} \qquad \boxed{NP_1 = 0.13} \text{ piles/ft}$$

$$NP_2 = \begin{cases} \dfrac{1}{P_2} & \text{if } P_2 > 0 \\ 0 & \text{otherwise} \end{cases} \qquad \boxed{NP_2 = 0.13} \text{ piles/ft}$$

$$NP_3 = \begin{cases} \dfrac{1}{P_3} & \text{if } P_3 > 0 \\ 0 & \text{otherwise} \end{cases} \qquad \boxed{NP_3 = 0.13} \text{ piles/ft}$$

$$NP = NP_1 + NP_2 + NP_3 \qquad \boxed{NP = 0.38} \text{ piles/ft}$$

Compute the centroid of the pile group

$$yy = \begin{cases} \dfrac{y_{p1}\, NP_1 + y_{p2}\, NP_2 + y_{p3}\, NP_3}{NP} & \text{if } NP > 0 \\ 0 & \text{otherwise} \end{cases} \qquad \boxed{yy = 4.08} \text{ ft}$$

Compute the distance from the centroid to the pile row

$$d_{p1} = yy - y_{p1} \qquad \boxed{d_{p1} = 2.83} \text{ ft}$$

$$d_{p2} = yy - y_{p2} \qquad \boxed{d_{p2} = 0.08} \text{ ft}$$

$$d_{p3} = yy - y_{p3} \qquad \boxed{d_{p3} = -2.92} \text{ ft}$$

Compute the section modulus for each of the pile rows

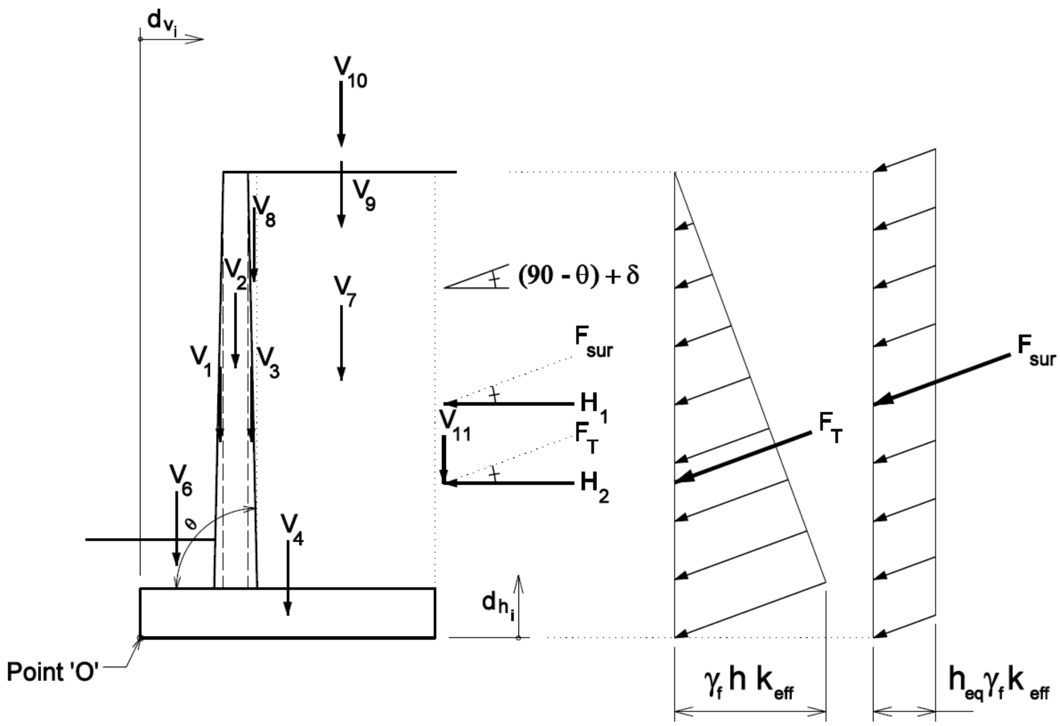
$$Sxx_1 = \frac{NP_1\, d_{p1}^{\,2} + NP_2\, d_{p2}^{\,2} + NP_3\, d_{p3}^{\,2}}{d_{p1}} \qquad \boxed{Sxx_1 = 0.73}$$

$$Sxx_2 = \frac{NP_1\, d_{p1}^{\,2} + NP_2\, d_{p2}^{\,2} + NP_3\, d_{p3}^{\,2}}{d_{p2}} \qquad \boxed{Sxx_2 = 24.81}$$

$$Sxx_3 = \frac{NP_1\, d_{p1}^{\,2} + NP_2\, d_{p2}^{\,2} + NP_3\, d_{p3}^{\,2}}{d_{p3}} \qquad \boxed{Sxx_3 = -0.71}$$

E14-4.4.3 Compute Unfactored Loads

The forces and moments are computed by using Figures E14-1.3-1 and E14-1.3-3 and by their respective load types **LRFD [Tables 3.4.1-1 and 3.4.1-2]**



<u>Figure E14-4.4-1</u>
CIP Concrete Wall - External Stability

Active Earth Force Resultant (kip/ft), F_T

$$F_T = \frac{1}{2} \gamma_f h^2 k_a$$ Active earth force resultant (EH) $\boxed{F_T = 10.17}$

Live Load Surcharge Load (kip/ft), F_{sur}

$$F_{sur} = \gamma_f h_{eq} h k_a$$ Live load surcharge resultant (LS) $\boxed{F_{sur} = 0.71}$

Vertical Loads (kip/ft), V_i

$$V_1 = \frac{1}{2} T_1 h' \gamma_c$$ Wall stem front batter (DC) $\boxed{V_1 = 0.72}$

$$V_2 = T_t h' \gamma_c$$ Wall stem (DC) $\boxed{V_2 = 3.23}$

$V_3 = \frac{1}{2} T_2\ h'\ \gamma_c$ Wall stem back batter (DC) $\boxed{V_3 = 1.44}$

$V_4 = D\ B\ \gamma_c$ Wall footing (DC) $\boxed{V_4 = 4.50}$

$V_6 = A\left(D_f - D\right)\gamma_{fd}$ Soil backfill - toe (EV) $\boxed{V_6 = 0.78}$

$V_7 = C\ h'\ \gamma_f$ Soil backfill - heel (EV) $\boxed{V_7 = 12.66}$

$V_8 = \frac{1}{2} T_2\ h'\ \gamma_f$ Soil backfill - batter (EV) $\boxed{V_8 = 1.16}$

$V_9 = \frac{1}{2}\left(T_2 + C\right)\left[\left(T_2 + C\right)\tan(\beta)\right]\gamma_f$

Soil backfill - backslope (EV) $\boxed{V_9 = 0.00}$

$V_{10} = h_{eq}\left(T_2 + C\right)\gamma_f$

Live load surcharge (LS) $\boxed{V_{10} = 0.58}$

$V_{11} = F_T\ \sin\left[\left(90\ deg - \theta\right) + \delta\right]$

Active earth force resultant (vertical component - EH) $\boxed{V_{11} = 3.38}$

Moments produced from vertical loads about Point 'O' (kip-ft/ft), MV_i

Moment Arm (ft)		Moment (kip-ft/ft)	
$d_{v1} = A + \frac{2}{3} T_1$	$\boxed{d_{v1} = 5.0}$	$MV_1 = V_1\ d_{v1}$	$\boxed{MV_1 = 3.6}$
$d_{v2} = A + T_1 + \frac{T_t}{2}$	$\boxed{d_{v2} = 5.7}$	$MV_2 = V_2\ d_{v2}$	$\boxed{MV_2 = 18.4}$
$d_{v3} = A + T_1 + T_t + \frac{T_2}{3}$	$\boxed{d_{v3} = 6.5}$	$MV_3 = V_3\ d_{v3}$	$\boxed{MV_3 = 9.4}$
$d_{v4} = \frac{B}{2}$	$\boxed{d_{v4} = 6.0}$	$MV_4 = V_4\ d_{v4}$	$\boxed{MV_4 = 27.0}$
$d_{v6} = \frac{A}{2}$	$\boxed{d_{v6} = 2.4}$	$MV_6 = V_6\ d_{v6}$	$\boxed{MV_6 = 1.9}$

$$d_{v7} = B - \frac{C}{2} \qquad \boxed{d_{v7} = 9.5} \qquad MV_7 = V_7\, d_{v7} \qquad \boxed{MV_7 = 120.8}$$

$$d_{v8} = A + T_1 + T_t + \frac{2T_2}{3} \qquad \boxed{d_{v8} = 6.8} \qquad MV_8 = V_8\, d_{v8} \qquad \boxed{MV_8 = 7.9}$$

$$d_{v9} = A + T_1 + T_t + \frac{2\left(T_2 + C\right)}{3} \qquad \boxed{d_{v9} = 10.1} \qquad MV_9 = V_9\, d_{v9} \qquad \boxed{MV_9 = 0.0}$$

$$d_{v10} = B - \left(\frac{T_2 + C}{2}\right) \qquad \boxed{d_{v10} = 9.1} \qquad MV_{10} = V_{10}\, d_{v10} \qquad \boxed{MV_{10} = 5.3}$$

$$d_{v11} = B \qquad \boxed{d_{v11} = 12.0} \qquad MV_{11} = V_{11}\, d_{v11} \qquad \boxed{MV_{11} = 40.5}$$

Horizontal Loads (kip/ft), H_i

$$H_1 = F_{sur} \cos\left[\left(90\ \mathrm{deg} - \theta\right) + \delta\right]$$ Live load surcharge (LS) $\boxed{H_1 = 0.67}$

$$H_2 = F_T \cos\left[\left(90\ \mathrm{deg} - \theta\right) + \delta\right]$$ Active earth force (horizontal component) (EH) $\boxed{H_2 = 9.59}$

Moments produced from horizontal loads about about Point 'O' (kip-ft/ft), MH_i

Moment Arm (ft)		Moment (kip-ft/ft)	
$d_{h1} = \frac{h}{2}$	$\boxed{d_{h1} = 12.0}$	$MH_1 = H_1\, d_{h1}$	$\boxed{MH_1 = 8.0}$
$d_{h2} = \frac{h}{3}$	$\boxed{d_{h2} = 8.0}$	$MH_2 = H_2\, d_{h2}$	$\boxed{MH_2 = 76.8}$

Summary of Unfactored Forces & Moments:

Load			Moment Arm		Moment		LRFD Load Type
Item	Description	Value (kip/ft)	Item	Value (ft)	Item	Value (kip-ft/ft)	
V_1	Wall stem front batter	0.72	d_{v1}	5.0	MV_1	3.6	DC
V_2	Wall stem	3.23	d_{v2}	5.7	MV_2	18.4	DC
V_3	Wall stem back batter	1.44	d_{v3}	6.5	MV_3	9.4	DC
V_4	Wall footing	4.50	d_{v4}	6.0	MV_4	27.0	DC
V_6	Soil backfill - Toe	0.78	d_{v6}	2.4	MV_6	1.9	EV
V_7	Soil backfill - Heel	12.66	d_{v7}	9.5	MV_7	120.8	EV
V_8	Soil backfill - Batter	1.16	d_{v8}	6.8	MV_8	7.9	EV
V_9	Soil backfill - Backslope	0.00	d_{v9}	10.1	MV_9	0.0	EV
V_{10}	Live load surcharge	0.58	d_{v10}	9.1	MV_{10}	5.3	LS
V_{11}	Active earth pressure	3.38	d_{v11}	12.0	MV_{11}	40.5	EH

Table E14-4.4-1
Unfactored Vertical Forces & Moments

Load			Moment Arm		Moment		LRFD Load Type
Item	Description	Value (kip/ft)	Item	Value (ft)	Item	Value (kip-ft/ft)	
H_1	Live load surcharge	0.67	d_{h1}	12.0	MH_1	8.0	LS
H_2	Active earth force	9.59	d_{h2}	8.0	MH_2	76.8	EH

Table E14-4.4-2
Unfactored Horizontal Forces & Moments

E14-4.4.4 Summarize Applicable Load and Resistance Factors

Maximum and minimum load factors shall be used to determine the extreme load effects. WisDOT's policy is to set all the load modifiers to zero($n = 1.0$). Factored loads and moments for each limit state are calculated by applying the appropriate load factors **LRFD [Tables 3.4.1-1 and 3.4.1-2]**. The following load combinations will be used in this example:

Load Combination	γ_{DC}	γ_{EV}	γ_{LS_V}	γ_{LS_H}	γ_{EH}	Application
Strength Ia	0.90	1.00	-	1.75	1.50	Sliding, Eccentricity
Strength Ib	1.25	1.35	1.75	1.75	1.50	Bearing, Wall Strength
Service I	1.00	1.00	1.00	1.00	1.00	Wall Crack Control

Table E14-4.4-3
Load Combinations

Load Combination Assumptions:

- Live load surcharge stabilizing loads (if applicable) are ignored for overturning and sliding analyses. Live load surcharge is used to compute maximum bearing pressure, wall strength and overall (global) stability.
- Minimum horizontal earth pressure ,$\gamma_{EH(min)} = 0.9$, will not control in this example based on B/H and lateral load inclination, but should be checked.
- Component load factors shall remain consistent throughout calculations. For example, the active earth force resultant (F_T) can be broken into component forces of either $V_{10}\gamma_{EH(max)}$ and $H_2\gamma_{EH(max)}$ or $V_{10}\gamma_{EH(min)}$ and $H_2\gamma_{EH(min)}$, not $V_{10}\gamma_{EH(min)}$ and $H_2\gamma_{EH(max)}$.

The loads discussed and tabulated previously can now be factored by the appropriate load factors and combined to determine the governing limit states for each design check.

E14-4.4.5 Compute Factored Loads and Moments

Unfactored loads by load type (kip/ft)

$V_{DC} = V_1 + V_2 + V_3 + V_4$ $\boxed{V_{DC} = 9.9}$

$V_{EV} = V_6 + V_7 + V_8 + V_9$ $\boxed{V_{EV} = 14.6}$

$V_{LS} = V_{10}$ $\boxed{V_{LS} = 0.6}$

$V_{EH} = V_{11}$ $\boxed{V_{EH} = 3.4}$

$H_{LS} = H_1$ $\boxed{H_{LS} = 0.7}$

$H_{EH} = H_2$ $\boxed{H_{EH} = 9.6}$

Unfactored moments by load type (kip-ft/ft)

$M_{DC} = MV_1 + MV_2 + MV_3 + MV_4$ $\boxed{M_{DC} = 58.4}$

$M_{EV} = MV_6 + MV_7 + MV_8 + MV_9$ $\boxed{M_{EV} = 130.6}$

$M_{LS1} = MV_{10}$ $\boxed{M_{LS1} = 5.3}$

$M_{EH1} = MV_{11}$ $\boxed{M_{EH1} = 40.5}$

$M_{LS2} = MH_1$ $\boxed{M_{LS2} = 8.0}$

$M_{EH2} = MH_2$ $\boxed{M_{EH2} = 76.8}$

Factored vertical loads by limit state (kip/ft)

$V\_Ia = n\left(0.90V_{DC} + 1.00V_{EV} + 0.00\ V_{LS} + 1.50\ V_{EH}\right)$ $\boxed{V\_Ia = 28.6}$

$V\_Ib = n\left(1.25V_{DC} + 1.35V_{EV} + 1.75\ V_{LS} + 1.50\ V_{EH}\right)$ $\boxed{V\_Ib = 38.2}$

$V\_Ser = n\left(1.00V_{DC} + 1.00V_{EV} + 1.00\ V_{LS} + 1.00\ V_{EH}\right)$ $\boxed{V\_Ser = 28.4}$

Factored horizontal loads by limit state (kip/ft)

$H\_Ia = n\left(1.75H_{LS} + 1.50H_{EH}\right)$ $\boxed{H\_Ia = 15.6}$

$H\_Ib = n\left(1.75H_{LS} + 1.50H_{EH}\right)$ $\boxed{H\_Ib = 15.6}$

$H\_Ser = n\left(1.00H_{LS} + 1.00H_{EH}\right)$ $\boxed{H\_Ser = 10.3}$

Factored moments produced by vertical Loads by limit state (kip-ft/ft)

$$MV\_Ia = n(0.90M_{DC} + 1.00M_{EV} + 0.00M_{LS1} + 1.50\ M_{EH1}) \qquad \boxed{MV\_Ia = 243.9}$$

$$MV\_Ib = n(1.25M_{DC} + 1.35M_{EV} + 1.75M_{LS1} + 1.50\ M_{EH1}) \qquad \boxed{MV\_Ib = 319.3}$$

$$MV\_Ser = n(1.00M_{DC} + 1.00M_{EV} + 1.00M_{LS1} + 1.00\ M_{EH1}) \qquad \boxed{MV\_Ser = 234.8}$$

Factored moments produced by horizontal loads by limit state (kip-ft/ft)

$$MH\_Ia = n(1.75M_{LS2} + 1.50\ M_{EH2}) \qquad \boxed{MH\_Ia = 129.1}$$

$$MH\_Ib = n(1.75M_{LS2} + 1.50\ M_{EH2}) \qquad \boxed{MH\_Ib = 129.1}$$

$$MH\_Ser = n(1.00M_{LS2} + 1.00\ M_{EH2}) \qquad \boxed{MH\_Ser = 84.8}$$

Load Combination	Vert. Loads V (kips/ft)	Moments MV (kip-kip/ft)	Horiz. Loads H (kips/ft)	Moments MH (kip-kip/ft)
Strength Ia	28.6	243.9	15.6	129.1
Strength Ib	38.2	319.3	15.6	129.1
Service I	28.4	234.8	10.3	84.8

Table E14-4.4-4
Summary of Factored Loads & Moments

E14-4.5 Evaluate Pile Reactions

Calculated loads for each limit state:

Strength Ia	Strength Ib	Service	
$V\_Ia = 28.56$	$V\_Ib = 38.15$	$V\_Ser = 28.45$	Vertical Load, kip/ft
$H\_Ia = 15.56$	$H\_Ib = 15.56$	$H\_Ser = 10.26$	Horizontal Load, kip/ft
$MV\_Ia = 243.90$	$MV\_Ib = 319.27$	$MV\_Ser = 234.76$	Moments (Vertical) kip-ft/ft
$MH\_Ia = 129.13$	$MH\_Ib = 129.13$	$MH\_Ser = 84.75$	Moments (Horizontal), kip-ft/ft

Compute the eccentricity about Point 'O'

$$e_{toe\_Ia} = \frac{MH\_Ia - MV\_Ia}{V\_Ia}$$ Strength Ia $\boxed{e_{toe\_Ia} = -4.02}$ ft

$$e_{toe\_Ib} = \frac{MH\_Ib - MV\_Ib}{V\_Ib}$$ Strength Ib $\boxed{e_{toe\_Ib} = -4.98}$ ft

$$e_{toe\_Ser} = \frac{MH\_Ser - MV\_Ser}{V\_Ser}$$ Service $\boxed{e_{toe\_Ser} = -5.27}$ ft

Compute the eccentricity about the neutral axis of the pile group

$$e_{NA\_Ia} = yy + e_{toe\_Ia}$$ Strength Ia $\boxed{e_{NA\_Ia} = 0.07}$ ft

$$e_{NA\_Ib} = yy + e_{toe\_Ib}$$ Strength Ib $\boxed{e_{NA\_Ib} = -0.90}$ ft

$$e_{NA\_Ser} = yy + e_{toe\_Ser}$$ Service $\boxed{e_{NA\_Ser} = -1.19}$ ft

Compute the moment about the neutral axis of the pile group

$$M_{NA\_Ia} = V\_Ia\ e_{NA\_Ia}$$ Strength Ia $\boxed{M_{NA\_Ia} = 1.9}$ kip-ft/ft

$$M_{NA\_Ib} = V\_Ib\ e_{NA\_Ib}$$ Strength Ib $\boxed{M_{NA\_Ib} = -34.4}$ kip-ft/ft

$$M_{NA\_Ser} = V\_Ser\ e_{NA\_Ser}$$ Service $\boxed{M_{NA\_Ser} = -33.9}$ kip-ft/ft

Compute the pile reactions for each limit state

Strength Ia

$$P_{U1a} = \frac{V\_Ia}{NP} + \frac{M_{NA\_Ia}}{Sxx_1}$$ $\boxed{P_{U1a} = 78.7}$ kip/pile

$$P_{U2a} = \frac{V\_Ia}{NP} + \frac{M_{NA\_Ia}}{Sxx_2}$$ $\boxed{P_{U2a} = 76.2}$ kip/pile

$$P_{U3a} = \frac{V\_Ia}{NP} + \frac{M_{NA\_Ia}}{Sxx_3}$$ $\boxed{P_{U3a} = 73.5}$ kip/pile

Strength Ib

$$P_{U1b} = \frac{V\_Ib}{NP} + \frac{M_{NA\_Ib}}{Sxx_1}$$ $\boxed{P_{U1b} = 54.6}$ kip/pile

$$P_{U2b} = \frac{V\_Ib}{NP} + \frac{M_{NA\_Ib}}{Sxx_2}$$ $\boxed{P_{U2b} = 100.4}$ kip/pile

$$P_{U3b} = \frac{V\_Ib}{NP} + \frac{M_{NA\_Ib}}{Sxx_3}$$ $\boxed{P_{U3b} = 150.2}$ kip/pile

Service

$$P_{U1\_Ser} = \frac{V\_Ser}{NP} + \frac{M_{NA\_Ser}}{Sxx_1}$$ $\boxed{P_{U1\_Ser} = 29.5}$ kip/pile

$$P_{U2\_Ser} = \frac{V\_Ser}{NP} + \frac{M_{NA\_Ser}}{Sxx_2}$$ $\boxed{P_{U2\_Ser} = 74.5}$ kip/pile

$$P_{U3\_Ser} = \frac{V\_Ser}{NP} + \frac{M_{NA\_Ser}}{Sxx_3}$$ $\boxed{P_{U3\_Ser} = 123.6}$ kip/pile

Load Combination	Row 1 (kip/pile)	Row 2 (kip/pile)	Row 3 (kip/pile)
Strength Ia	78.7	76.2	73.5
Strength Ib	54.6	100.4	150.2
Service I	29.5	74.5	123.6

Table E14-4.5-1
Summary of Factored Pile Reactions (Vertical)

E14-4.6 Evaluate External Stability of Wall

Three potential external failure mechanisms will be considered in this example. These failures include pile bearing resistance, limiting eccentricity and lateral resistance. Global (overall) stability requirements are assumed to have been satisfied in prior calculations. Design calculations will be carried out for the governing limit states only.

E14-4.6.1 Pile Bearing Resistance

Axial and lateral pile capacities from Geotechnical Site Investigation Report:

$Pile\_Axial = 220$ Pile axial capacity, kips

$pile\_batter = 4$ Pile batter (pile_batter V:1H)

$H_{r1} = 11.00$ Battered pile row 1 lateral capacity, kips/pile

$H_{r2} = 11.00$ Battered pile row 2 lateral capacity, kips/pile

$H_{r3} = 14.00$ Vertical pile row 3 lateral capacity, kips/pile

Determine the horizontal and vertical components of the battered pile

$$pile\_angle = atan\left(\frac{1}{pile\_batter}\right) \qquad \boxed{pile\_angle = 14.0\,deg}$$

$$P_{Rb\_H} = Pile\_Axial\ \sin(pile\_angle) \qquad \boxed{P_{Rb\_H} = 53.4} \text{ kips/pile}$$

$$P_{Rb\_V} = Pile\_Axial\ \cos(pile\_angle) \qquad \boxed{P_{Rb\_V} = 213.4} \text{ kips/pile}$$

Calculate axial capacity of battered piles

$$P_R = P_{Rb\_V} \qquad \boxed{P_R = 213.4} \text{ kips/pile}$$

$$P_u = \max\left(P_{U1a}, P_{U2a}, P_{U1b}, P_{U2b}\right) \qquad \boxed{P_u = 100.4} \text{ kips/pile}$$

$$CDR_{Brg\_B\_Pile} = \frac{P_R}{P_u} \qquad \boxed{CDR_{Brg\_B\_Pile} = 2.13}$$

Is the $CDR \ge 1.0$? $\boxed{check = \text{"OK"}}$

Calculate axial capacity of vertical piles

$$P_R = Pile\_Axial \qquad \boxed{P_R = 220.0}$$

$$P_u = \max\left(P_{U3a}, P_{U3b}\right) \qquad \boxed{P_u = 150.2}$$

$$CDR_{Brg\_V\_Pile} = \frac{P_R}{P_u} \qquad \boxed{CDR_{Brg\_V\_Pile} = 1.46}$$

Is the $CDR \ge 1.0$? $\boxed{check = \text{"OK"}}$

E14-4.6.2 Pile Sliding Resistance

For sliding failure, the horizontal force effects, H_u, is checked against the sliding resistance, H_R, where $H_R=\phi H_n$ **LRFD [10.6.3.4]**. The following calculations are based on **Strength Ia**:

Factored Lateral Force, H_u

$H_u = H\_Ia$ $\boxed{H_u = 15.6}$ kip/ft

Sliding Resistance, H_R

It is assumed that the P-y method was used for the pile analysis (LPILE), thus group effects shall be considered. Calculate sliding capacity of the effective pile group per **LRFD [Table-10.7.2.4-1]:**

$B_{yy} = 11.78$ Depth of pile, in

$$\frac{PS1 + PS2}{\frac{B_{yy}}{12}} = 5.86$$ Say:5B

Note: It was assumed that pile row 1 and 3 are aligned throughout the pile group and that pile row 2 will not effect the lateral pile group resistance. Pile row 1 and 3 will then be applied row 1 and 2 "5B" multipliers, respectfully.

"5B" Pile multipliers

$row1 = 1.00$

$row2 = 1.00$

$row3 = 0.80$

Lateral group resistance

$H_{R1} = row1\ H_{r1}\ NP_1 + row2\ H_{r2}\ NP_2 + row3\ H_{r3}\ NP_3$ $\boxed{H_{R1} = 4.15}$ kip/ft

Batter resistance

$H_{R2} = P_{Rb\_H}\left(NP_1 + NP_2\right)$ $\boxed{H_{R2} = 13.34}$ kip/ft

Compute factored resistance against failure by sliding, R_R

$H_R = H_{R1} + H_{R2}$ $\boxed{H_R = 17.49}$ kip/ft

Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} = \frac{H_R}{H_u}$$ $\boxed{CDR_{Sliding} = 1.12}$

Is the $CDR \geq 1.0$? $\boxed{check = \text{"OK"}}$

E14-4.7 Evaluate Wall Structural Design

Note: CIP concrete walls are a non-proprietary wall system and the structural design computations shall be performed by the wall designer.

Wall structural design computations for shear and flexure will be considered in this example. Crack control and temperature and shrinkage considerations will also be included.

E14-4.7.1 Evaluate Wall Footing

E14-4.7.1.1 Evalute One-Way Shear

Design for one-way shear in only the transverse direction.

Compute the effective shear depth,d_v, for the heel:

$cover = 2.0$ in

$s = 9.0$ in (bar spacing)

$Bar_{No} = 7$ (transverse bar size)

$Bar_D = 0.500$ in (transverse bar diameter)

$Bar_A = 0.600$ in² (transverse bar area)

$$A_{s\_heel} = \frac{Bar_A}{\frac{s}{12}} \qquad \boxed{A_{s\_heel} = 0.80} \text{ in}^2\text{/ft}$$

$$d_{s\_heel} = D\,12 - cover - \frac{Bar_D}{2} \qquad \boxed{d_{s\_heel} = 27.8} \text{ in}$$

$$a\_heel = \frac{A_{s\_heel}\, f_y}{0.85\, f'_c\, b} \qquad \boxed{a\_heel = 1.3} \text{ in}$$

$$d_{v1} = d_{s\_heel} - \frac{a\_heel}{2} \qquad \boxed{d_{v1} = 27.1} \text{ in}$$

$$d_{v2} = 0.9\, d_{s\_heel} \qquad \boxed{d_{v2} = 25.0} \text{ in}$$

$$d_{v3} = 0.72\, D\, 12 \qquad \boxed{d_{v3} = 21.6} \text{ in}$$

$$d_{v\_heel} = \max(d_{v1}, d_{v2}, d_{v3}) \qquad \boxed{d_{v\_heel} = 27.1} \text{ in}$$

Compute the effective shear depth,d_v, for the toe

$cover = 6.0$ in

$s = 9.0$ in (bar spacing)

$Bar_{No} = 7$ (transverse bar size)

$Bar_D = 0.88$ in (transverse bar diameter)

$Bar_A = 0.60$ in^2 (transverse bar area)

$$A_{s\_toe} = \frac{Bar_A}{\frac{s}{12}} \qquad \boxed{A_{s\_toe} = 0.80} \text{ in}^2\text{/ft}$$

$$d_{s\_toe} = D\ 12 - cover - \frac{Bar_D}{2} \qquad \boxed{d_{s\_toe} = 23.6} \text{ in}$$

$$a\_toe = \frac{A_{s\_toe}\ f_y}{0.85\ f'_c\ b} \qquad \boxed{a\_toe = 1.3} \text{ in}$$

$$d_{v1} = d_{s\_toe} - \frac{a\_toe}{2} \qquad \boxed{d_{v1} = 22.9} \text{ in}$$

$$d_{v2} = 0.9\ d_{s\_toe} \qquad \boxed{d_{v2} = 21.2} \text{ in}$$

$$d_{v\_toe} = \max(d_{v1}, d_{v2}) \qquad \boxed{d_{v\_toe} = 22.9} \text{ in}$$

Determine the distance from Point 'O' to the critical sections:

$$y\_crit\_toe = A\ 12 - d_{v\_toe} \qquad \boxed{y\_crit\_toe = 34.1} \text{ in}$$

$$y\_crit\_heel = B\ 12 - C\ 12 + d_{v\_heel} \qquad \boxed{y\_crit\_heel = 112.2} \text{ in}$$

Determine the distance from Point 'O' to the pile limits:

$$y_{v1\_neg} = y_{p1}\ 12 - \frac{B_{yy}}{2} \qquad \boxed{y_{v1\_neg} = 9.1} \text{ in}$$

$$y_{v1\_pos} = y_{p1}\ 12 + \frac{B_{yy}}{2} \qquad \boxed{y_{v1\_pos} = 20.9} \text{ in}$$

$$y_{v2\_neg} = y_{p2}\ 12 - \frac{B_{yy}}{2} \qquad \boxed{y_{v2\_neg} = 42.1} \text{ in}$$

$$y_{v2\_pos} = y_{p2}\,12 + \frac{B_{yy}}{2}$$ $\boxed{y_{v2\_pos} = 53.9}$ in

$$y_{v3\_neg} = y_{p3}\,12 - \frac{B_{yy}}{2}$$ $\boxed{y_{v3\_neg} = 78.1}$ in

$$y_{v3\_pos} = y_{p3}\,12 + \frac{B_{yy}}{2}$$ $\boxed{y_{v3\_pos} = 89.9}$ in

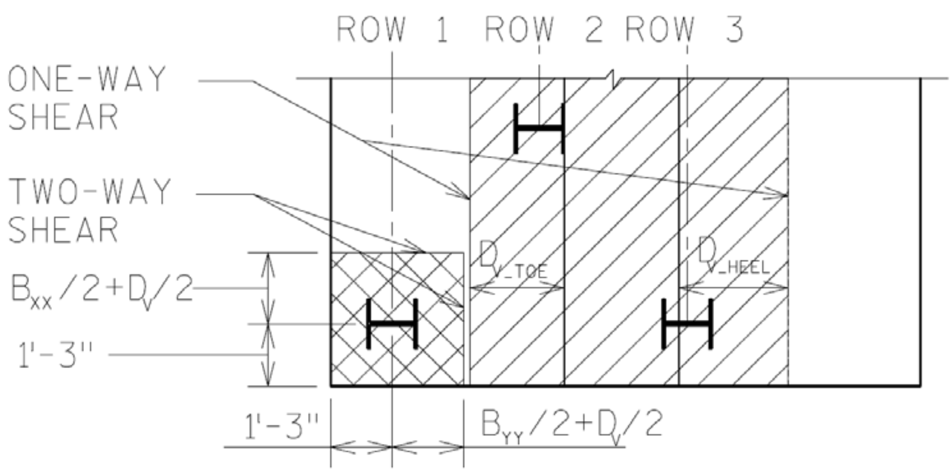


Figure E14-4.7-1
Partial Footing Plan for Critical Shear Sections

Determine if the pile rows are "Outside", "On", or "Inside" the critical sections

Since the pile row 1 falls "Outside" the critical sections, the full row pile reaction will be used for shear

$$P_{U1} = \max\left(P_{U1a}, P_{U1b}\right)$$ $\boxed{P_{U1} = 78.7}$ kip

$$V_{u\_Pile1} = 1.0\left(P_{U1}\,NP_1\right)$$ $\boxed{V_{u\_Pile1} = 9.8}$ kip/ft

Since the pile row 2 and 3 falls "Inside" the critical sections, none of the row pile reactions will be used for shear

The load applied to the critical section is based on the proportion of the piles located outside of the critical toe or heel section. In this case, pile row 1 falls outside the toe critical section and the full row pile reaction will be used for shear.

$V_u = V_{u\_Pile1}$ $\boxed{V_u = 9.8}$ 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]**

Nominal one-way action shear resistance for structures without transverse reinforcement, V_n, is taken as the lesser of V_{n1} and V_{n2}

$\beta = 2.0$

$V_c = 0.0316\ \beta \sqrt{f'_c}\ b\ d_{v\_toe}$

$V_{n1} = V_c$ $\boxed{V_{n1} = 32.5}$ kip/ft

$V_{n2} = 0.25\ f'_c\ b\ d_{v\_toe}$ $\boxed{V_{n2} = 240.3}$ kip/ft

$V_n = \min(V_{n1}, V_{n2})$ $\boxed{V_n = 32.5}$ kip/ft

$\phi_v = 0.90$

$V_r = \phi_v\ V_n$ $\boxed{V_r = 29.2}$ kip/ft

$\boxed{V_u = 9.8}$ kip/ft

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

E14-4.7.1.2 Evaluate Two-Way Shear

For two-way action around the maximum loaded pile, the pile critical perimeter, b_o, is located a minimum of $0.5d_v$ from the perimeter of the pile. If portions of the critical perimeter are located off the footing, that portion of the critical perimeter is limited by the footing edge.

Two-way action should be checked for the maximum loaded pile.

$V_u = \max(P_{U1a}, P_{U2a}, P_{U3a}, P_{U1b}, P_{U2b}, P_{U3b})$ $\boxed{V_u = 150.2}$ kip

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

$$V_n = \min\left(V_{n1}, V_{n2}\right)$$ $\boxed{V_n = 349.7}$ 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:

$$M_n = A_{s\_heel} f_y \left(d_{s\_heel} - \frac{a\_heel}{2}\right)\frac{1}{12}$$ $\boxed{M_n = 108.3}$ 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

$$\phi_F = \begin{cases} 0.75 \ \text{ if } \ \dfrac{d_{s}\_heel}{c} < \dfrac{5}{3} \\ 0.65 + 0.15\left(\dfrac{d_{s}\_heel}{c} - 1\right) \ \text{ if } \ \dfrac{5}{3} \le \dfrac{d_{s}\_heel}{c} \le \dfrac{8}{3} \\ 0.90 \ \text{ otherwise} \end{cases}$$

$\boxed{\phi_F = 0.90}$

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:

$M_r = \phi_F M_n$ $\boxed{M_r = 97.5}$ 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]**:

$f_r = 0.37\sqrt{f'_c}$ $\boxed{f_r = 0.692}$ ksi

$I_g = \dfrac{1}{12} b (D\ 12)^3$ $\boxed{I_g = 27000}$ in^4

$y_t = \dfrac{1}{2} D\ 12$ $\boxed{y_t = 15.00}$ in

$S_c = \dfrac{I_g}{y_t}$ $\boxed{S_c = 1800}$ in^3

$M_{cr} = S_c f_r \dfrac{1}{12}$ $\boxed{M_{cr} = 103.8}$ kip-ft/ft

$\boxed{1.2\ M_{cr} = 124.6}$ kip-ft/ft

$\boxed{1.33\ M_u = 88.2}$ kip-ft/ft

Is M_r greater than the lesser value of 1.2*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:

$M_u = \max(M_{u\_Ia}, M_{u\_Ib})$ $\boxed{M_u = 41.6}$ 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

$$\phi_F = \begin{cases} 0.75 & \text{if } \frac{d_{s\_toe}}{c} < \frac{5}{3} \\ 0.65 + 0.15\left(\frac{d_{s\_toe}}{c} - 1\right) & \text{if } \frac{5}{3} \le \frac{d_{s\_toe}}{c} \le \frac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$ $\boxed{\phi_F = 0.90}$

Calculate the flexural factored resistance, M_r:

$M_r = \phi_F M_n$ $\boxed{M_r = 82.4}$ kip-ft/ft

$\boxed{M_u = 41.6}$ kip-ft/ft

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

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

$f_r = 0.37\sqrt{f'_c}$ $\boxed{f_r = 0.692}$ ksi

$I_g = \frac{1}{12} b (D\ 12)^3$ $\boxed{I_g = 27000}$ in^4

$y_t = \frac{1}{2} D\ 12$ $\boxed{y_t = 15.00}$ in

$S_c = \frac{I_g}{y_t}$ $\boxed{S_c = 1800}$ in^3

$M_{cr} = S_c f_r \frac{1}{12}$ $\boxed{M_{cr} = 103.8}$ kip-ft/ft

$\boxed{1.2\ M_{cr} = 124.6}$ kip-ft/ft

$\boxed{1.33\ M_u = 55.3}$ kip-ft/ft

Is M_r greater than the lesser value of $1.2*M_{cr}$ and $1.33*M_u$?

$\boxed{check = "OK"}$

E14-4.7.1.5 Evaluate Longitudinal Reinforcement Strength

The structural design of the longitudinal reinforcement, assuming the footing acts as a continuous beam over pile supports, is calculated using the maximum pile reactions.

Compute the effective shear depth, d_v ,for the longitudinal reinforcement

$cover = 6.0$ in

$s = 12.0$ in (bar spacing)

$Bar_{No} = 5$ (longitudinal bar size)

$Bar_D = 0.625$ in (longitudinal bar diameter)

$Bar_A = 0.310$ in^2 (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:

$$M_n = A_{s\_long}\, f_y \left(d_s - \frac{a\_long}{2}\right) \frac{1}{12} \qquad \boxed{M_n = 35.0} \text{ kip-ft/ft}$$

Calculate the flexural resistance factor ϕ_F:

$$\beta_1 = 0.85$$

$$c = \frac{a\_toe}{\beta_1}$$ $\boxed{c = 1.58}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \frac{d_s}{c} < \frac{5}{3} \\ 0.65 + 0.15\left(\frac{d_s}{c} - 1\right) & \text{if } \frac{5}{3} \le \frac{d_s}{c} \le \frac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$ $\boxed{\phi_F = 0.90}$

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]**:

$$f_r = 0.37\sqrt{f'_c}$$ $\boxed{f_r = 0.692}$ ksi

$$I_g = \frac{1}{12} b (D\ 12)^3$$ $\boxed{I_g = 27000}$ in^4

$$y_t = \frac{1}{2} D\ 12$$ $\boxed{y_t = 15.00}$ in

$$S_c = \frac{I_g}{y_t}$$ $\boxed{S_c = 1800}$ in^3

$$M_{cr} = S_c f_r \frac{1}{12}$$ $\boxed{M_{cr} = 103.8}$ kip-ft/ft

$\boxed{1.2\ M_{cr} = 124.6}$ kip-ft/ft

$\boxed{1.33\ M_u = 27.1}$ kip-ft/ft

Is M_r greater than the lesser value of $1.2*M_{cr}$ and $1.33*M_u$?

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

E14-4.7.2 Evaluate Stem Strength

Unfactored Stem Horizontal Loads and Moments:

$H_1 = \gamma_f\, h_{eq}\, h'\, k_a \cos(90\ deg - \theta + \delta)$ $\boxed{H_1 = 0.6}$ kip/ft

$H_2 = \frac{1}{2} \gamma_f\, h'^2\, k_a \cos(90\ deg - \theta + \delta)$ $\boxed{H_2 = 7.7}$ kip/ft

$M_1 = H_1 \left(\frac{h'}{2}\right)$ $\boxed{M_1 = 6.4}$ kip-ft/ft

$M_2 = H_2 \left(\frac{h'}{3}\right)$ $\boxed{M_2 = 55.2}$ kip-ft/ft

Factored Stem Horizontal Loads and Moments:

for **Strength Ib**:

$H_{u1} = 1.75\, H_1 + 1.50\, H_2$ $\boxed{H_{u1} = 12.6}$ kip/ft

$M_{u1} = 1.75\, M_1 + 1.50\, M_2$ $\boxed{M_{u1} = 94.0}$ kip-ft/ft

for **Service I**:

$H_{u3} = 1.00\, H_1 + 1.00\, H_2$ $\boxed{H_{u3} = 8.3}$ kip/ft

$M_{u3} = 1.00\, M_1 + 1.00\, M_2$ $\boxed{M_{u3} = 61.6}$ kip-ft/ft

E14-4.7.2.1 Evaluate Stem Shear Strength at Footing

$V_u = H_{u1}$ $\boxed{V_u = 12.6}$ kip/ft

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.8.3.3]**

$V_{n1} = V_c$ **LRFD [Eq 5.8.3.3-1]**

where: $V_c = 0.0316\, \beta \sqrt{f'_c}\, b_v\, d_v$

$V_{n2} = 0.25\, f'_c\, b_v\, d_v$ **LRFD [Eq 5.8.3.3-2]**

Compute the shear resistance due to concrete, V_c :

$cover = 2.0$ in

$s = 12.0$ in (bar spacing)

$Bar_{No} = 9$ (transverse bar size)

$Bar_D = 1.13$ in (transverse bar diameter)

$Bar_A = 1.00$ in^2 (transverse bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}}$$ $\boxed{A_s = 1.00}$ in^2/ft

$$d_s = T_b\ 12 - cover - \frac{Bar_D}{2}$$ $\boxed{d_s = 25.6}$ in

$$a = \frac{A_s\ f_y}{0.85\ f'_c\ b}$$ $\boxed{a = 1.7}$ in

$$d_{v1} = d_s - \frac{a}{2}$$ $\boxed{d_{v1} = 24.7}$ in

$$d_{v2} = 0.9\ d_s$$ $\boxed{d_{v2} = 23.0}$ in

$$d_{v3} = 0.72\ T_b\ 12$$ $\boxed{d_{v3} = 20.3}$ in

$$d_v = \max(d_{v1}, d_{v2}, d_{v3})$$ $\boxed{d_v = 24.7}$ in

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

$\beta = 2.0$

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

$$V_{n1} = V_c$$ $\boxed{V_{n1} = 35.1}$ kip/ft

$$V_{n2} = 0.25\ f'_c\ b\ d_v$$ $\boxed{V_{n2} = 259.6}$ kip/ft

$$V_n = \min(V_{n1}, V_{n2})$$ $\boxed{V_n = 35.1}$ kip/ft

$$V_r = \phi_v\ V_n$$ $\boxed{V_r = 31.6}$ kip/ft

$\boxed{V_u = 12.6}$ kip/ft

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

E14-4.7.2.2 Evaluate Stem Flexural Strength at Footing

$$M_u = M_{u1}$$ $\boxed{M_u = 94.0}$ kip-ft/ft

Calculate the capacity of the stem in flexure at the face of the footing:

$$M_n = A_s\ f_y\left(d_s - \frac{a}{2}\right)\frac{1}{12}$$ $\boxed{M_n = 123.6}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$\beta_1 = 0.85$

$c = \dfrac{a}{\beta_1}$ $\boxed{c = 1.98}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \dfrac{d_s}{c} < \dfrac{5}{3} \\ 0.65 + 0.15\left(\dfrac{d_s}{c} - 1\right) & \text{if } \dfrac{5}{3} \le \dfrac{d_s}{c} \le \dfrac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$

$\boxed{\phi_F = 0.90}$

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

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

$f_r = 0.37\sqrt{f'_c}$ $\boxed{f_r = 0.69}$ 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

$M_{cr\_s} = S_c\, f_r\, \dfrac{1}{12}$

$\boxed{M_{cr\_s} = 91.3}$ kip-ft/ft

$\boxed{1.2\, M_{cr\_s} = 109.5}$ kip-ft/ft

$\boxed{1.33\, M_u = 125.0}$ kip-ft/ft

Is M_r greater than the lesser value of $1.2*M_{cr}$ and $1.33*M_u$?

$\boxed{check = "OK"}$

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

$$\rho = \frac{A_s}{d_s\, b} \qquad \boxed{\rho = 0.00326}$$

$$n = \frac{E_s}{E_c} \qquad \boxed{n = 8.09}$$

$$k = \sqrt{(\rho\ n)^2 + 2\ \rho\ n} - \rho\ n \qquad \boxed{k = 0.205}$$

$$j = 1 - \frac{k}{3} \qquad \boxed{j = 0.932}$$

$$d_c = cover + \frac{Bar_D}{2} \qquad \boxed{d_c = 2.6}\ \text{in}$$

$$f_{ss} = \frac{M_{u3}}{A_s\ j\ d_s}\, 12 \qquad \boxed{f_{ss} = 31.0}\ \text{ksi}$$

$$h = T_b\ 12$$

$$\beta_s = 1 + \frac{d_c}{0.7\left(h - d_c\right)} \qquad \boxed{\beta_s = 1.1}$$

$\gamma_e = 1.00$ for Class 1 exposure

$$s_{max} = \frac{700\ \gamma_e}{\beta_s\ f_{ss}} - 2\ d_c \qquad \boxed{s_{max} = 14.6}\ \text{in}$$

$$\boxed{s = 12.0}\ \text{in}$$

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

E14-4.7.2.3 Transfer of Force at Base of Stem

Specification requires that the transfer of lateral forces from the stem to the footing be in accordance with the shear-transfer provisions of **LRFD [5.8.4]**. That calculation will not be presented. Refer to E13-1.9.3 for a similar computation.

E14-4.7.3 Temperature and Shrinkage Steel

E14-4.7.3.1 Temperature and Shrinkage Steel for Footing

The footing will not be exposed to daily temperature changes. Thus temperature and shrinkage steel is not required.

E14-4.7.3.2 Temperature and Shrinkage Steel of Stem

The stem will be exposed to daily temperature changes. In accordance with AASTHO **LRFD [5.10.8]** the stem shall provide temperature and shrinkage steel on each face and in each direction as calculated below:

$s = 18.0$ in (bar spacing)

$Bar_{No} = 4$ (bar size)

$Bar_A = 0.20$ in² (temperature and shrinkage bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}}$$ (temperature and shrinkage provided)

$\boxed{A_s = 0.13}$ in²/ft

$b_s = (H - D)\ 12$ least width of stem $\boxed{b_s = 258.0}$ in

$h_s = T_t\ 12$ least thickness of stem $\boxed{h_s = 12.0}$ in

$$A_{ts} = \frac{1.3\ b_s\ h_s}{2\left(b_s + h_s\right) f_y}$$ Area of reinforcement per foot, on each face and in each direction $\boxed{A_{ts} = 0.12}$ in²/ft

Is $0.11 \le A_s \le 0.60$? $\boxed{check = \text{"OK"}}$

Is $A_s > A_{ts}$? $\boxed{check = \text{"OK"}}$

Check the maximum spacing requirements

$s_1 = \min\left(3\ h_s, 18\right)$ $\boxed{s_1 = 18.0}$ in

$$s_2 = \begin{vmatrix} 12 & \text{if} \quad h_s > 18 \\ s_1 & \text{otherwise} \end{vmatrix}$$ For walls and footings (in) $\boxed{s_2 = 18.0}$ in

$s_{max} = \min\left(s_1, s_2\right)$ $\boxed{s_{max} = 18.0}$ in

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

E14-4.8 Summary of Results

E14-4.8.1 Summary of External Stability

Based on the defined project parameters the following external stability checks have been satisfied:

External Check	CDR
	Strength I
Bearing	1.46
Eccentricity	> 10
Sliding	1.12

Table E14-4.8-1
Summary of External Stability Computations

E14-4.8.2 Summary of Wall Strength Design

The required wall reinforcing from the previous computations are presented in Figure E14-6.9-1.

E14-4.8.3 Drainage Design

Drainage requirements shall be investigated and detailed accordingly. In this example drainage requirements are met by providing granular, free draining backfill materail with a pipe underdrain located at the bottom of the wall (Assumed wall is adjacent to sidewalk) as shown in Figure E14-6.9-1.

E14-4.9 Final Cast-In-Place Concrete Wall Schematic

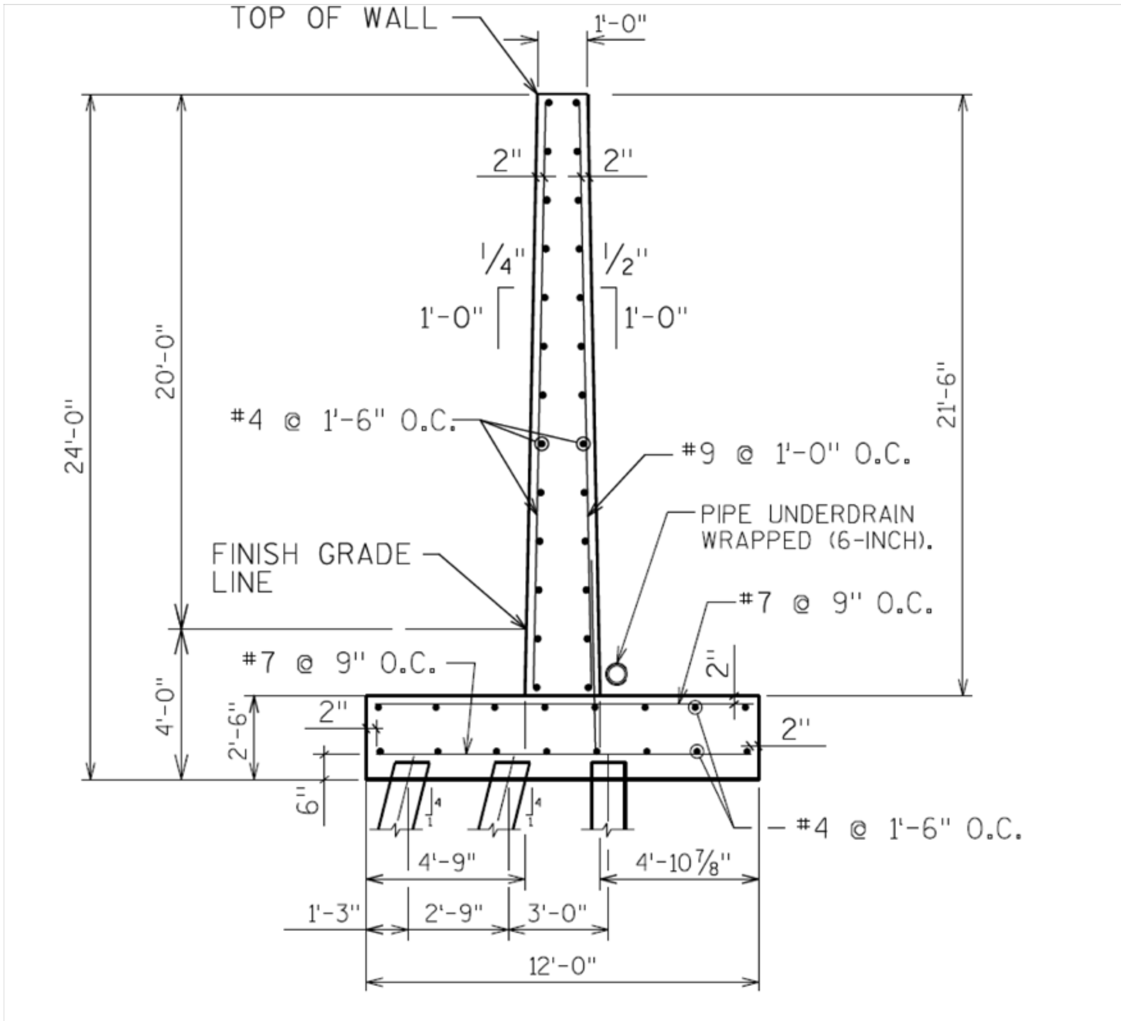


Figure E14-4.9-1
Cast-In-Place Wall Schematic

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E14-5 Sheet Pile Wall, LRFD

General

This example shows design calculations for permanent sheet pile walls conforming to the LRFD Bridge Design Specifications and the WisDOT Bridge Manual. *(Example is current through LRFD Fifth Edition - 2010)*

Sample design calculations for required embedment depth and determining preliminary design sections will be presented. The overall stability and settlement calculations will not be shown in this example, but are required.

Design steps presented in 14.10.5 are used for the wall design.

E14-5.1 Establish Project Requirements

The following example is for a permanent cantilever sheet pile wall penetrating sand and having the low water level at the dredge line as shown in Figure E14-5.1-1. External stability and structural components are the designer's (WisDOT/consultant) responsibility.

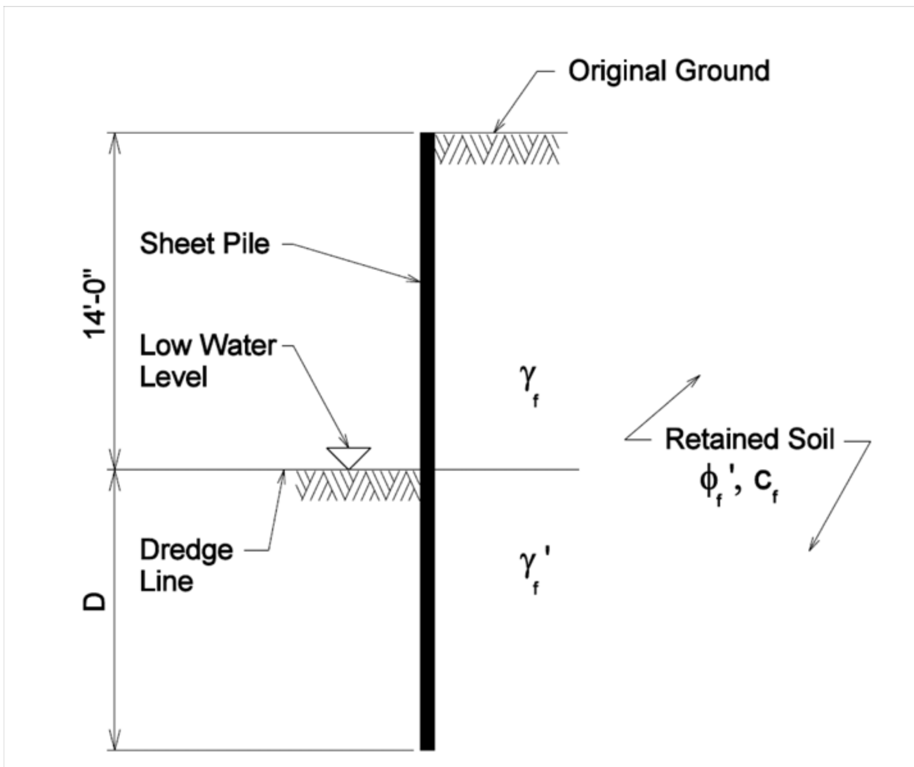


Figure E14-5.1-1
Cantilever Sheet Pile Wall with Horizontal Backslope

Wall Geometry

$H = 14$ Design wall height, ft

$\theta = 90\ \text{deg}$ Angle of back face of wall to horizontal

$\beta = 0\ \text{deg}$ Inclination of ground slope behind face of wall (horizontal)

E14-5.2 Design Parameters

Project Parameters

$\text{Design\_Life} = 75$ Wall design life (min), years **LRFD [11.5.1]**

Soil Properties (From Geotechnical Site Investigation Report)

Designer to determine if long-term or short-term soil strength parameters govern external stability.

Soil Design Parameters

$\phi_f = 35\ \text{deg}$ Angle of internal friction

$\gamma = 0.115$ Unit weight of soil, kcf

$\gamma_w = 0.0624$ Unit weight of water, kcf

$\gamma' = \gamma - \gamma_w$ Effective unit weight of soil, kcf

$\boxed{\gamma' = 0.053}$

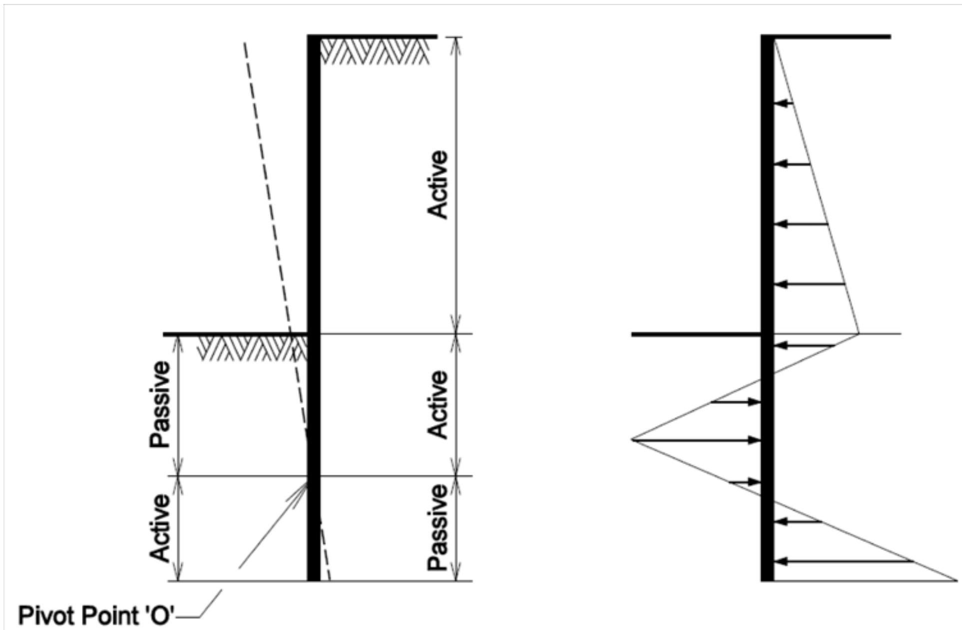
$c = 0\ \text{psf}$ Cohesion, psf

Live Load Surcharge Parameters

$SUR = 0.100$ Live load surcharge for walls without traffic, ksf (14.4.5.4.2)

E14-5.3 Establish Earth Pressure Diagram

In accordance with **LRFD [3.11.5.6]** "simplified" and "conventional" methods may be used for lateral earth pressure distributions. This example will use the "simplified" method as shown in **LRFD [Figure 3.11.5.3-2]**. The "conventional" method would result in a more exact solution and is based on Figure E14-5.3-1(b) lateral load distributions.



(a) (b)

Figure E14-5.3-1
Cantilever Sheet Pile Wall Penetrating a Sand Layer: (a) Wall Yielding Pattern and Earth Pressure Zones; (b) Conventional Net Earth Pressure Distribution (After Das, 2007).

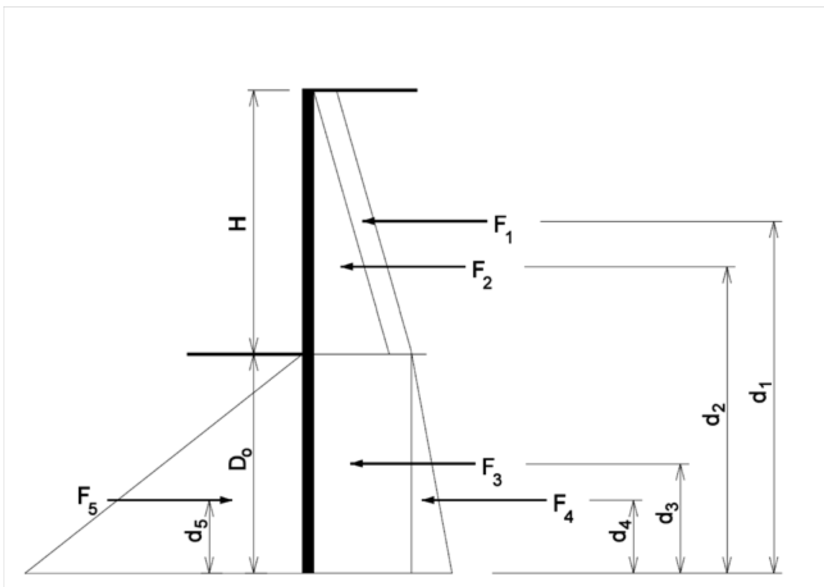


Figure E14-5.3-2
Cantilever Sheet Pile Wall Free-Body Diagram - Simplified Method

E14-5.4 Permanent and Transient Loads

In this example, horizontal earth pressures 'EH' will be used as shown in Figure E14-5.3-1(b). For simplicity, no transient, vertical or surcharge loads are present in this example.

E14-5.4.1 Compute Active Earth Pressure

Compute the coefficient of active earth pressure using Rankine Theory

$$\phi_f = 35\,\text{deg}$$

$$k_a = \tan\left(45\,\text{deg} - \frac{\phi_f}{2}\right)^2$$

$$\boxed{k_a = 0.271}$$

E14-5.4.2 Compute Passive Earth Pressure

Compute the coefficient of passive earth pressure using Rankine Theory

$$\phi_f = 35\,\text{deg}$$

$$k_p = \tan\left(45\,\text{deg} + \frac{\phi_f}{2}\right)^2$$

$$\boxed{k_p = 3.690}$$

E14-5.4.3 Compute Factored Loads

The active earth pressure is factored by its appropriate LRFD load type 'EH' **LRFD [Tables 3.4.1-1 and 3.4.1-2]**. Where as the passive earth pressure is factored by its appropriate resistance factor **LRFD [Table 11.5.6-1]**.

Compute the factored active earth pressure coefficient, K_a

$k_a = 0.271$	Unfactored active earth pressure coefficient	
$\gamma_{EH} = 1.50$	Horizontal earth pressure load factor (maximum)	
$K_a = \gamma_{EH}\,k_a$	Factored active earth pressure coefficient	$\boxed{K_a = 0.406}$

Compute the factored passive earth pressure coefficient, K_p

$k_p = 3.69$	Unfactored passive earth pressure coefficient	
$\phi_p = 0.75$	Nongravity cantilevered wall resistance factored for flexural capacity of a vertical element **LRFD [Table 11.5.6-1]**	
$K_p = \phi_p\,k_p$	Factored passive earth pressure coefficient	$\boxed{K_p = 2.768}$

E14-3.5 Compute Wall Embedment Depth and Factored Bending Moment

Compute the required embedment depth, D_o, corresponding to the depth where the factored active and passive moments are in equilibrium from Figure E14-5.3-2. Trial-and-error is used to determine the depth by adjusting D_o in the following equations:

$D_o = 27.5$ ft

Force (factored)

$F_1 = -(K_a\ SUR)\ H$ $\boxed{F_1 = -0.57}$ kip/ft

$F_2 = \frac{-1}{2}(\gamma\ K_a\ H)\ H$ $\boxed{F_2 = -4.58}$ kip/ft

$F_3 = -(\gamma\ K_a\ H + K_a\ SUR)\ D_o$ $\boxed{F_3 = -19.11}$ kip/ft

$F_4 = \frac{-1}{2}(\gamma'\ K_a\ D_o)\ D_o$ $\boxed{F_4 = -8.08}$ kip/ft

$F_5 = \frac{1}{2}(\gamma'\ K_p\ D_o)\ D_o$ $\boxed{F_5 = 55.05}$ kip/ft

Moment Arm			Moment (factored)		
$d_1 = \frac{H}{2} + D_o$	$\boxed{d_1 = 34.5}$	ft	$M_1 = F_1\ d_1$	$\boxed{M_1 = -19.6}$	kip-ft/ft
$d_2 = \frac{H}{3} + D_o$	$\boxed{d_2 = 32.2}$	ft	$M_2 = F_2\ d_2$	$\boxed{M_2 = -147.4}$	kip-ft/ft
$d_3 = \frac{D_o}{2}$	$\boxed{d_3 = 13.8}$	ft	$M_3 = F_3\ d_3$	$\boxed{M_3 = -262.8}$	kip-ft/ft
$d_4 = \frac{D_o}{3}$	$\boxed{d_4 = 9.2}$	ft	$M_4 = F_4\ d_4$	$\boxed{M_4 = -74.1}$	kip-ft/ft
$d_5 = \frac{D_o}{3}$	$\boxed{d_5 = 9.2}$	ft	$M_5 = F_5\ d_5$	$\boxed{M_5 = 504.6}$	kip-ft/ft

$\Sigma M = M_1 + M_2 + M_3 + M_4 + M_5$ (Approximately equal to zero) $\boxed{\Sigma M = 0.66}$ kip-ft/ft

Capacity:Demand Ratio (CDR) at D_o

$M_a = M_1 + M_2 + M_3 + M_4$ Factored active moments $\boxed{M_a = -503.9}$ kip-ft/ft

$M_p = M_5$ Factored passive moments $\boxed{M_p = 504.6}$ kip-ft/ft

$CDR = \left|\frac{M_p}{M_a}\right|$ $\boxed{CDR = 1.00}$

Is the $CDR \geq 1.0$? $\boxed{check = \text{"OK"}}$

Compute the required embedment depth, D. Since the wall embedment depth uses the Simplified Method with continuous vertical elements a 20% increase in embedment will be included as shown in **LRFD [Figure 3.11.5.6-3]**.

$D = 1.2\ D_o$ $\boxed{D = 33.00}$ ft

Compute the location of the maximum bending moment, M_{max}, corresponding to the depth where the factored active and passive lateral forces are in equilibrium from Figure E14-5.3-2. Trial-and-error is used to determine the depth by adjusting D_o in the following equations:

$D_o = 16.3$ ft

Force (factored)

$F_1 = -(K_a\ SUR)\ H$ $\boxed{F_1 = -0.57}$ kip/ft

$F_2 = \frac{-1}{2}(\gamma\ K_a\ H)\ H$ $\boxed{F_2 = -4.58}$ kip/ft

$F_3 = -(\gamma\ K_a\ H + K_a\ SUR)\ D_o$ $\boxed{F_3 = -11.33}$ kip/ft

$F_4 = \frac{-1}{2}(\gamma'\ K_a\ D_o)\ D_o$ $\boxed{F_4 = -2.84}$ kip/ft

$F_5 = \frac{1}{2}(\gamma'\ K_p\ D_o)\ D_o$ $\boxed{F_5 = 19.34}$ kip/ft

$\Sigma F = F_1 + F_2 + F_3 + F_4 + F_5$ (Approximately equal to zero) $\boxed{\Sigma F = 0.02}$ kip-ft/ft

Moment Arm		Moment (factored)	
$d_1 = \frac{H}{2} + D_o$	$\boxed{d_1 = 23.3}$ ft	$M_1 = F_1\ d_1$	$\boxed{M_1 = -13.3}$ kip-ft/ft
$d_2 = \frac{H}{3} + D_o$	$\boxed{d_2 = 21.0}$ ft	$M_2 = F_2\ d_2$	$\boxed{M_2 = -96.1}$ kip-ft/ft
$d_3 = \frac{D_o}{2}$	$\boxed{d_3 = 8.2}$ ft	$M_3 = F_3\ d_3$	$\boxed{M_3 = -92.3}$ kip-ft/ft
$d_4 = \frac{D_o}{3}$	$\boxed{d_4 = 5.4}$ ft	$M_4 = F_4\ d_4$	$\boxed{M_4 = -15.4}$ kip-ft/ft
$d_5 = \frac{D_o}{3}$	$\boxed{d_5 = 5.4}$ ft	$M_5 = F_5\ d_5$	$\boxed{M_5 = 105.1}$ kip-ft/ft

$\Sigma M = M_1 + M_2 + M_3 + M_4 + M_5$ $\boxed{\Sigma M = -112.0}$ kip-ft/ft

$M_{max} = |\Sigma M|$ $\boxed{M_{max} = 112.0}$ kip-ft/ft

Figure E14-5.5-1 tabulates the above computations in a spreadsheet for varying embedment depths.

D_o	F_1	F_2	F_3	F_4	F_5	d_1	d_2	d_3	d_4	d_5	Fa	Fp	Fa+Fp	M_1	M_2	M_3	M_4	M_5	M_a	M_p	CDR	M_a+M_p
0	-0.6	-4.6	0.0	0.0	0.0	7.0	4.7	0.0	0.0	0.0	-5.2	0.0	-5.2	-4	-21	0	0	0	-25	0	0.0	-25.4
2	-0.6	-4.6	-1.4	0.0	0.3	9.0	6.7	1.0	0.7	0.7	-6.6	0.3	-6.3	-5	-31	-1	0	0	-37	0	0.0	-36.9
4	-0.6	-4.6	-2.8	-0.2	1.2	11.0	8.7	2.0	1.3	1.3	-8.1	1.2	-6.9	-6	-40	-6	0	2	-52	2	0.0	-50.2
6	-0.6	-4.6	-4.2	-0.4	2.6	13.0	10.7	3.0	2.0	2.0	-9.7	2.6	-7.1	-7	-49	-13	-1	5	-70	5	0.1	-64.3
8	-0.6	-4.6	-5.6	-0.7	4.7	15.0	12.7	4.0	2.7	2.7	-11.4	4.7	-6.7	-9	-58	-22	-2	12	-91	12	0.1	-78.2
10	-0.6	-4.6	-7.0	-1.1	7.3	17.0	14.7	5.0	3.3	3.3	-13.2	7.3	-5.9	-10	-67	-35	-4	24	-115	24	0.2	-90.9
12	-0.6	-4.6	-8.3	-1.5	10.5	19.0	16.7	6.0	4.0	4.0	-15.0	10.5	-4.5	-11	-76	-50	-6	42	-143	42	0.3	-101.4
14	-0.6	-4.6	-9.7	-2.1	14.3	21.0	18.7	7.0	4.7	4.7	-17.0	14.3	-2.7	-12	-86	-68	-10	67	-175	67	0.4	-108.8
16.3	**-0.6**	**-4.6**	**-11.3**	**-2.8**	**19.3**	**23.3**	**21.0**	**8.2**	**5.4**	**5.4**	**-19.3**	**19.3**	**0.0**	**-13**	**-96**	**-92**	**-15**	**105**	**-217**	**105**	**0.5**	**-112.0**
18	-0.6	-4.6	-12.5	-3.5	23.6	25.0	22.7	9.0	6.0	6.0	-21.1	23.6	2.5	-14	-104	-113	-21	142	-251	142	0.6	-110.0
20	-0.6	-4.6	-13.9	-4.3	29.1	27.0	24.7	10.0	6.7	6.7	-23.3	29.1	5.8	-15	-113	-139	-29	194	-296	194	0.7	-101.8
22	-0.6	-4.6	-15.3	-5.2	35.2	29.0	26.7	11.0	7.3	7.3	-25.6	35.2	9.6	-17	-122	-168	-38	258	-345	258	0.7	-86.5
24	-0.6	-4.6	-16.7	-6.2	41.9	31.0	28.7	12.0	8.0	8.0	-28.0	41.9	13.9	-18	-131	-200	-49	335	-398	335	0.8	-63.0
26	-0.6	-4.6	-18.1	-7.2	49.2	33.0	30.7	13.0	8.7	8.7	-30.4	49.2	18.8	-19	-140	-235	-63	426	-457	426	0.9	-30.4
27.5	**-0.6**	**-4.6**	**-19.1**	**-8.1**	**54.9**	**34.5**	**32.1**	**13.7**	**9.2**	**9.2**	**-32.3**	**54.9**	**22.6**	**-20**	**-147**	**-262**	**-74**	**503**	**-503**	**503**	**1.0**	**0.0**
30	-0.6	-4.6	-20.9	-9.6	65.5	37.0	34.7	15.0	10.0	10.0	-35.6	65.5	29.9	-21	-159	-313	-96	655	-589	655	1.1	66.2
32	-0.6	-4.6	-22.2	-10.9	74.5	39.0	36.7	16.0	10.7	10.7	-38.3	74.5	36.2	-22	-168	-356	-117	795	-663	795	1.2	132.2

Results Tabulated Above Values		
Required Embedment Depth, D_o (M_p/M_a>1)=	**27.47**	ft
Actual Embedment (1.2*D_o) =	**32.96**	ft
Maximum Factored Moment Location (F_a+F_p=0) =	**16.30**	ft
Maximum Factored Design Moment=	**112.0**	kip-ft/ft

Figure E14-5.5-1
Design Analysis for Cantilever Sheet Pile Wall

E14-5.6 Compute the Required Flexural Resistance

The following is a design check for flexural resistance:

$$M_{max} \le \phi_f M_n \qquad \phi_f M_n = \phi_f F_y Z$$

$M_{max} = 112.0$ kip-ft/ft

$\phi_f = 0.90$ Resistance factor for flexure (based on nongravity cantilevered walls for the flexural capacity of vertical elements **LRFD [Table 11.5.6-1]**)

M_n Nominal flexural resistance of the section

$F_y = 50$ Steel yield stress, ksi (assumed A572 Grade 50)

Z Plastic section modulus (in³/ft)

$$Z_{reqd} = \frac{M_{max}\ 12}{\phi_f\ F_y} \qquad \boxed{Z_{reqd} = 29.87} \text{ in}^3\text{/ft}$$

Based on this minimum section modulus a preliminary sheet pile section PZ-27 (Z=36.49 in³/ft) is selected. Additional design checks shall be made based on project requirements.

E14-5.7 Final Sheet Pile Wall Schematic

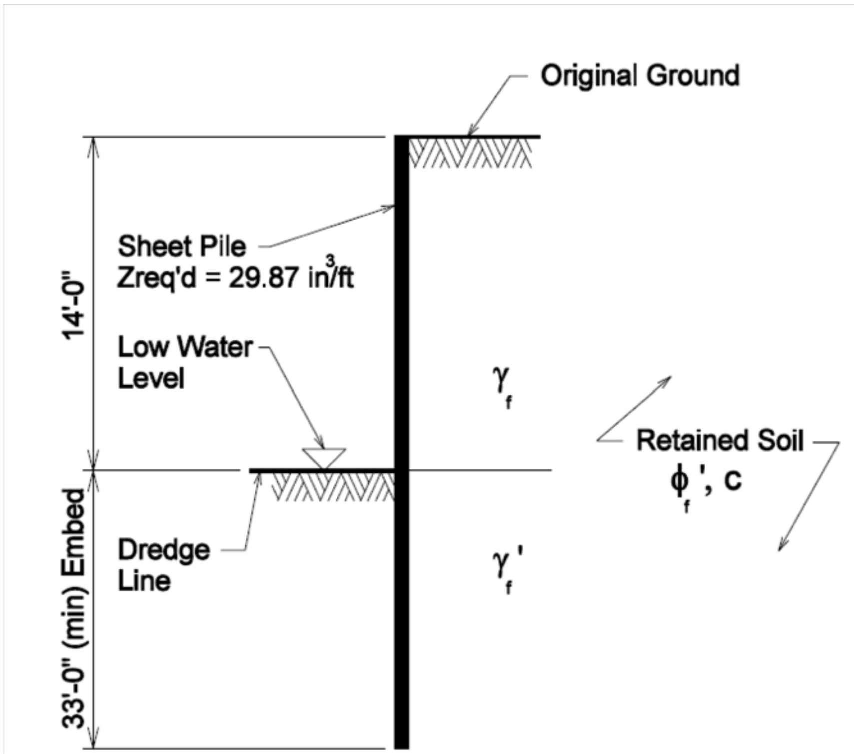


Figure E14-5.7-1
Cantilever Sheet Pile Wall Schematic

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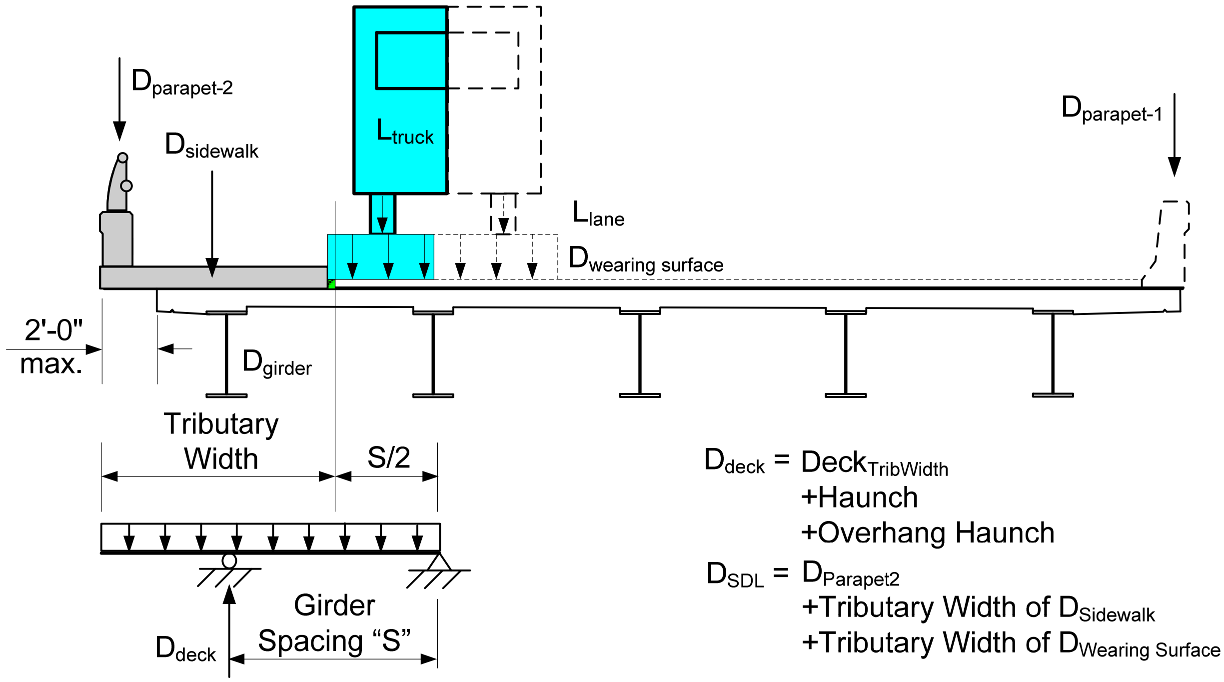


Use Tributary Width for Deck and Superimposed Dead Loads

Figure 17.2-17
Distribution of Loads to Exterior Girder for Girder Structure with Raised Sidewalk
Design Case 1

The distribution of loads to the exterior girder for Design Case 1 is calculated as follows:

For dead loads, the exterior girder must resist the girder weight (girder self-weight and any other miscellaneous weight applied to the girder), deck weight (including the concrete haunch and overhang haunch) and all superimposed dead loads. The distribution of the deck weight and all superimposed dead loads to the exterior girder are based on the tributary width, as shown in the previous figure.

For the live load, the live load distribution factor for Design Case 1 is based only on the application of the lever rule. It is recommended for Design Case 1 lane load to use the same distribution factor as for the truck load. The appropriate multiple presence factor of 1.2 must be applied.

The general equation for loads applied to the exterior girder is as follows:

$$\text{Total Load} = D_{girder} + D_{deck} + D_{superimposed\ \ DL} + \left[(DF_{ext})(L_{truck} + L_{lane})\right]$$

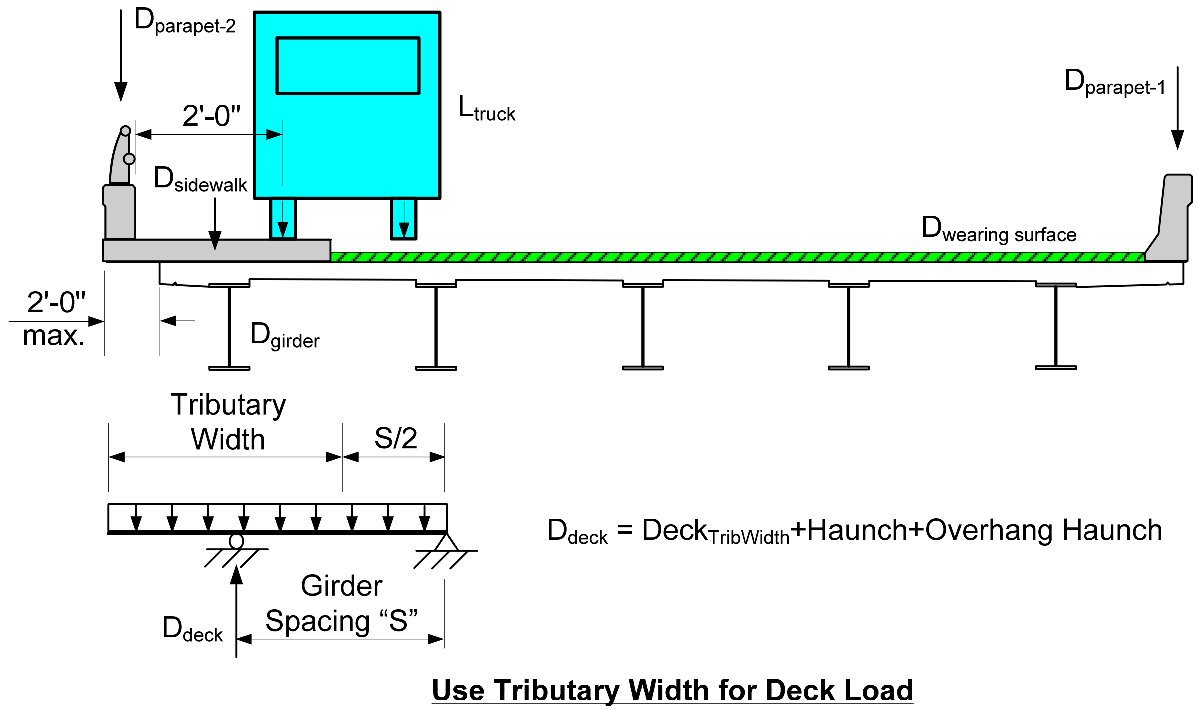


Figure 17.2-18
Distribution of Loads to Exterior Girder for Girder Structure with Raised Sidewalk
Design Case 2

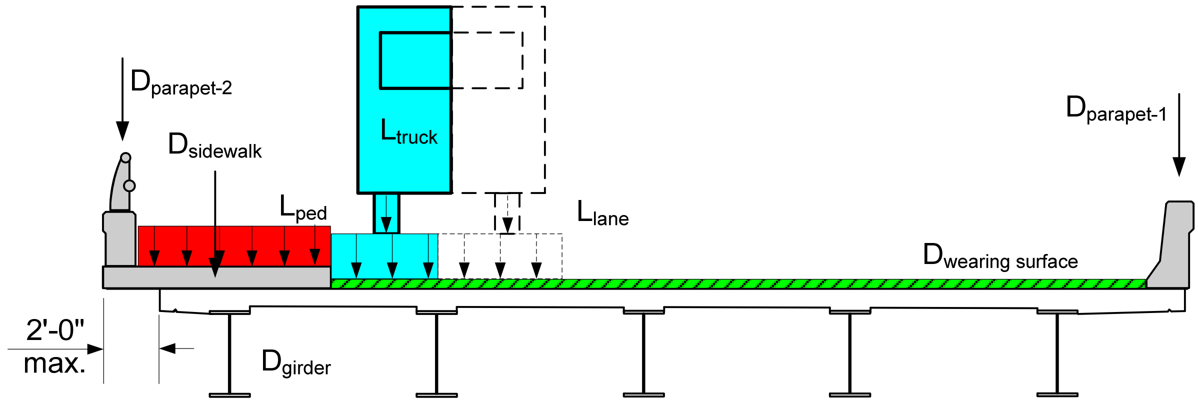
The distribution of loads to the exterior girder for Design Case 2 is calculated as follows:

For dead loads, the exterior girder must resist the girder weight (girder self-weight and any other miscellaneous weight applied to the girder), deck weight (including the concrete haunch and overhang haunch) and all superimposed dead loads. The distribution of the deck weight to the exterior girder is based on the tributary width, as shown in the previous figure. The distribution of all superimposed dead loads is based on equal distribution to all girders.

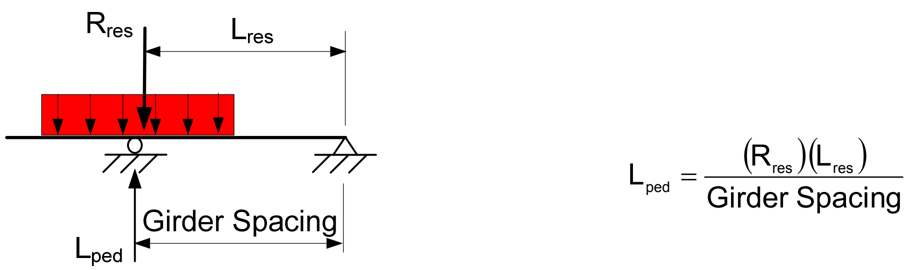
For the live load, the live load distribution factor for Design Case 2 is based only on the application of the lever rule. The appropriate multiple presence factor of 1.2 must be applied.

The general equation for loads applied to the exterior girder is as follows:

$$\text{Total Load} = D_{\text{girder}} + D_{\text{deck}} + \left(\frac{D_{\text{wearing surface}} + D_{\text{sidewalk}} + D_{\text{parapet-1}} + D_{\text{parapet-2}}}{\text{No. of Girders}} \right) + \left[\left(DF_{\text{ext}} \right) \left(L_{\text{truck}} \right) \right]$$



Use Tributary Width for Deck Load



Apply Lever Rule for Pedestrian Load

Figure 17.2-19
Distribution of Loads to Exterior Girder for Girder Structure with Raised Sidewalk
Design Case 3

The distribution of loads to the exterior girder for Design Case 3 is calculated as follows:

For dead loads, the exterior girder must resist the girder weight (girder self-weight and any other miscellaneous weight applied to the girder), deck weight (including the concrete haunch and overhang haunch) and all superimposed dead loads. The distribution of the deck weight to the exterior girder is based on the tributary width, as shown in the previous figure. The distribution of all superimposed dead loads is based on equal distribution to all girders.

For the live load, the live load distribution factor for Design Case 3 is based only on the application of the lever rule. It is recommended for Design Case 3 lane load to use the same distribution factor as for the truck load. The appropriate multiple presence factor of 1.0 must be applied.

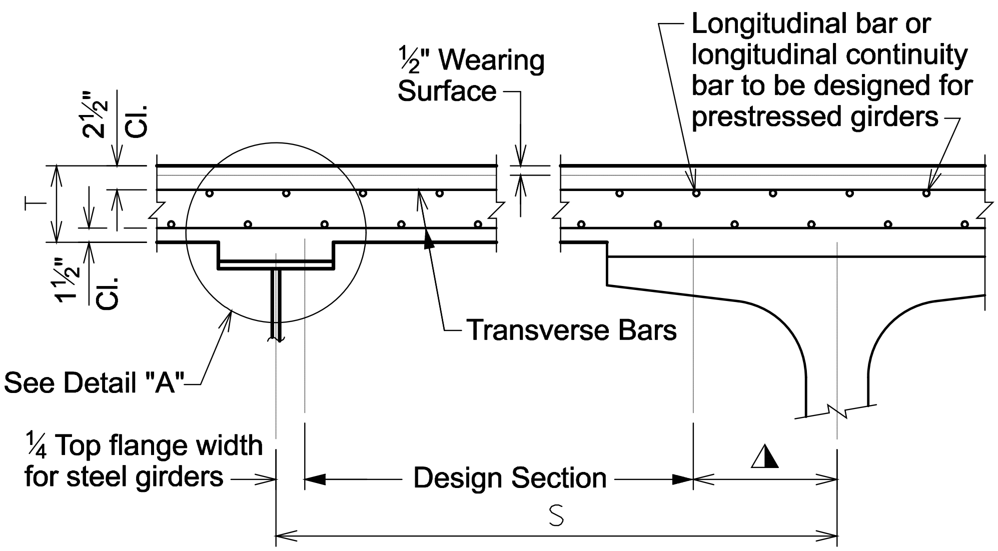
For pedestrian loads, the distribution to the exterior girder is based on the lever rule, as shown in the previous figure.

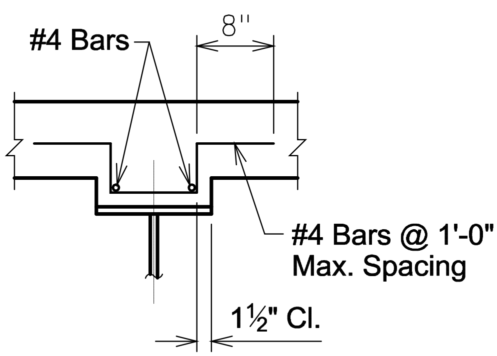
The general equation for loads applied to the exterior girder is as follows:

$$\text{Total Load} = D_{\text{girder}} + D_{\text{deck}} + \left(\frac{D_{\text{wearing surface}} + D_{\text{sidewalk}} + D_{\text{parapet-1}} + D_{\text{parapet-2}}}{\text{No. of Girders}} \right) + L_{\text{ped}} + \left[(DF_{\text{ext}})(L_{\text{truck}} + L_{\text{lane}}) \right]$$

The distance from the centerline of the girder to the design section is computed in accordance with **LRFD [4.6.2.1.6]**. For steel beams, this distance is equal to one-quarter of the flange width from the centerline of support. For prestressed concrete girders, this distance is equal to the values presented in Figure 17.5-1, along with bar locations and clearances.

Note: Transverse reinforcing steel requirements (bar size and spacing) are determined for both positive moment requirements and negative moment requirements, and the same reinforcing steel is used in both the top and bottom of slab as shown in Table 17.5-1 and Table 17.5-2. Longitudinal reinforcement in Table 17.5-3 and Table 17.5-4 is based on a percentage of the bottom transverse reinforcement required *by actual design calculations* (not a percentage of what is in the tables). **The tables should be used for deck reinforcement, with continuity bars in prestressed girder bridges being the only deck reinforcement requiring calculation.**





Detail "A"

Prestressed Girder	△ (Inches)
28"	6
36"	4
36W", 45W"	11
54W", 72W", 82W"	15
45"	5
54"	6
70"	10

Figure 17.5-1
Transverse Section thru Slab on Girders

For skews of 20° and under, place transverse bars along the skew. For skews greater than 20°, place transverse bars perpendicular to the girders.

Detail "A", as presented in Figure 17.5-1, should be used for decks when shear connectors extend less than 2 inches into the slab on steel girder bridges or 3 inches on prestressed concrete girder bridges.

Several transverse reinforcing steel tables are provided in this chapter. The reinforcing steel in Table 17.5-1 and Table 17.5-2 does not account for deck overhangs. However, the minimum amount of reinforcing steel required in the deck overhangs is presented in various design tables in 17.6.

The reinforcement shown in Table 17.5-1 and Table 17.5-2 is based on both the Strength I requirement and crack control requirement.

Crack control was checked in accordance with **LRFD [5.7.3.4]**. The bar spacing cannot exceed the value from the following formula:

$$s \leq \frac{700(\gamma)}{\beta_s f_s} - 2d_c$$

Where:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

γ = 0.75 for decks

β_s = Ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face

f_s = Tensile stress in reinforcement at the service limit state (ksi)

d_c = Top concrete cover less ½ inch wearing surface plus ½ bar diameter or bottom concrete cover plus ½ bar diameter (inches)

h = Slab depth minus ½ inch wearing surface (inches)

WisDOT policy item:

The thickness of the sacrificial ½-inch wearing surface shall not be included in the calculation of d_c.

Table 17.5-1 and Table 17.5-2 were developed for specified values of the distance from the centerline of girder to the design section for negative moment. Those specified values – 0, 3, 6, 9, 12 and 18 inches – were selected to match values used in **AASHTO [Table A4-1]**. For a girder in which the distance from the centerline of girder to the design section for negative moment is not included in Table 17.5-1 and Table 17.5-2, the engineer may interpolate between the closest two values in the tables or can use the more conservative of the two values.

Transverse Reinforcing Steel for Deck Slabs on Girders for New Bridges and Deck Replacements, HL-93 Loading, Slab Thickness "T" ≥ 8"							
Slab Thickness "T" (Inches)	Girder Spacing "S"	Distance from Centerline of Girder to Design Section					
		0"	3"	6"	9"	12"	18"
8	4'-6"	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7
8	4'-9"	#4 @ 6.5	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7
8	5'-0"	#4 @ 6.5	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7
8	5'-3"	#4 @ 6.5	#4 @ 6.5	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7
8	5'-6"	#4 @ 6.5	#4 @ 6.5	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7
8	5'-9"	#5 @ 8	#4 @ 6.5	#4 @ 6.5	#4 @ 7	#4 @ 7	#4 @ 7
8	6'-0"	#5 @ 7.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
8	6'-3"	#5 @ 7	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
8	6'-6"	#5 @ 7	#5 @ 8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
8	6'-9"	#5 @ 7	#5 @ 7.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
8	7'-0"	#5 @ 6.5	#5 @ 7	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5
8	7'-3"	#5 @ 6.5	#5 @ 7	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5
8	7'-6"	#5 @ 6.5	#5 @ 7	#5 @ 8	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5
8	7'-9"	#5 @ 6	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8
8	8'-0"	#6 @ 7.5	#5 @ 6.5	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8
8.5	8'-3"	#5 @ 6.5	#5 @ 7	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5
8.5	8'-6"	#5 @ 6.5	#5 @ 7	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5
8.5	8'-9"	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8
8.5	9'-0"	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8
8.5	9'-3"	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8
9	9'-6"	#5 @ 6.5	#5 @ 7	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
9	9'-9"	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8
9	10'-0"	#5 @ 6	#5 @ 6.5	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8
9	10'-3"	#6 @ 7	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8
9	10'-6"	#6 @ 7	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
9.5	10'-9"	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8
9.5	11'-0"	#6 @ 7	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8

9.5	11'-3"	#6 @ 7	#5 @ 6	#5 @ 6.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
9.5	11'-6"	#6 @ 7	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5
10	11'-9"	#6 @ 7	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8
10	12'-0"	#6 @ 7	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 8	#5 @ 8
10	12'-3"	#6 @ 7	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5
10	12'-6"	#6 @ 7	#6 @ 7	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5
10.5	12'-9"	#6 @ 7	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 8
10.5	13'-0"	#6 @ 7	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5
10.5	13'-3"	#6 @ 7	#6 @ 7	#5 @ 6.5	#5 @ 6.5	#5 @ 7.5	#5 @ 7.5
10.5	13'-6"	#6 @ 6.5	#6 @ 7	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5
11	13'-9"	#6 @ 7	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5
11	14'-0"	#6 @ 7	#6 @ 7	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5

Table 17.5-1

Transverse Reinforcing Steel for Deck Slabs on Girders
for New Bridges and Deck Replacements, HL-93 Loading, Slab Thickness "T" ≥ 8"

Transverse Reinforcing Steel for Deck Slabs on Girders for Deck Replacements, HL-93 Loading, Slab Thickness "T" < 8"							
Slab Thickness "T" (Inches)	Girder Spacing "S"	Distance from Centerline of Girder to Design Section					
		0"	3"	6"	9"	12"	18"
6.5	4'-0"	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
6.5	4'-3"	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
6.5	4'-6"	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
6.5	4'-9"	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
6.5	5'-0"	#5 @ 6.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
6.5	5'-3"	#5 @ 6	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7
6.5	5'-6"	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7
6.5	5'-9"	#6 @ 7	#5 @ 6	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7
6.5	6'-0"	#6 @ 6.5	#6 @ 7.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7
6.5	6'-3"	#6 @ 6.5	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7
6.5	6'-6"	#6 @ 6	#6 @ 7	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7
6.5	6'-9"	(1)	#6 @ 7	#5 @ 6	#5 @ 7	#5 @ 7	#5 @ 7
6.5	7'-0"	(1)	#6 @ 6.5	#6 @ 7.5	#5 @ 6.5	#5 @ 6.5	#5 @ 6.5
7	4'-0"	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8

7	4’-3”	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
7	4’-6”	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
7	4’-9”	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
7	5’-0”	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
7	5’-3”	#5 @ 7	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
7	5’-6”	#5 @ 6.5	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
7	5’-9”	#5 @ 6.5	#5 @ 7	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
7	6’-0”	#6 @ 7.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
7	6’-3”	#6 @ 7.5	#5 @ 6.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
7	6’-6”	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
7	6’-9”	#6 @ 7	#5 @ 6	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
7	7’-0”	#6 @ 6.5	#6 @ 7.5	#5 @ 6.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
7	7’-3”	#6 @ 6.5	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7
7	7’-6”	#6 @ 6	#6 @ 7	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7
7	7’-9”	(1)	#6 @ 6.5	#5 @ 6	#5 @ 7	#5 @ 7	#5 @ 7
7	8’-0”	(1)	#6 @ 6.5	#5 @ 6	#5 @ 7	#5 @ 7	#5 @ 7
7.5	4’-0”	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
7.5	4’-3”	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
7.5	4’-6”	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
7.5	4’-9”	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
7.5	5’-0”	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
7.5	5’-3”	#5 @ 8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
7.5	5’-6”	#5 @ 7.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
7.5	5’-9”	#5 @ 7	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5
7.5	6’-0”	#5 @ 7	#5 @ 8	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5
7.5	6’-3”	#5 @ 6.5	#5 @ 7.5	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5
7.5	6’-6”	#5 @ 6.5	#5 @ 7	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
7.5	6’-9”	#5 @ 6.5	#5 @ 7	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
7.5	7’-0”	#6 @ 7.5	#5 @ 6.5	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8
7.5	7’-3”	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 8	#5 @ 8	#5 @ 8
7.5	7’-6”	#6 @ 7	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
7.5	7’-9”	#6 @ 7	#5 @ 6	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
7.5	8’-0”	#6 @ 6.5	#6 @ 7.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
7.5	8’-3”	#6 @ 6.5	#6 @ 7.5	#5 @ 6.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
7.5	8’-6”	#6 @ 6.5	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7

7.5	8'-9"	#6 @ 6.5	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7
7.5	9'-0"	#6 @ 6.5	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7
7.5	9'-3"	#6 @ 6.5	#6 @ 7	#5 @ 6.5	#5 @ 6.5	#5 @ 6.5	#5 @ 6.5
7.5	9'-6"	#6 @ 6	#6 @ 6.5	#5 @ 6	#5 @ 6.5	#5 @ 6.5	#5 @ 6.5

(1) When these regions are encountered, the next thicker deck section shall be used.

Table 17.5-2

Transverse Reinforcing Steel for Deck Slabs on
Girders for Deck Replacements, HL-93 Loading, Slab Thickness "T" < 8"
(Only use Table 17.5-2 if Bridge Rating is unacceptable with "T" ≥ 8")

The transverse reinforcing steel presented in Table 17.5-1 and Table 17.5-2 is designed in accordance with *AASHTO LRFD*. The tables are developed based on deck concrete with a 28-day compressive strength of f'_c = 4 ksi and reinforcing steel with a yield strength of f_y = 60 ksi. However, the same tables should be used for concrete strength of 5 ksi.

The clearance for the top steel is 2 1/2", and the clearance for the bottom steel is 1 1/2". The dead load includes 20 psf for future wearing surface.

The reinforcing bars shown in the tables are for one layer only. Identical steel should be placed in both the top and bottom layers.

17.5.3.2 Longitudinal Reinforcement

The amount of bottom longitudinal reinforcement required is as specified in **LRFD [9.7.3.2]** and shown in Table 17.5-3 and Table 17.5-4. It is based on a percentage of the transverse reinforcing steel for positive moment. For the main reinforcement perpendicular to traffic, the percentage equals:

$$\frac{220}{\sqrt{S}} \leq 67\%$$

Where:

S = Girder spacing, as calculated based on Figure 17.5-1 (feet)

WisDOT exception to AASHTO:

The girder spacing shall be used in the equation above for calculating the percentage of transverse steel to be used as longitudinal reinforcement. This definition replaces the one stated in **LRFD [9.7.3.2]** to use the effective girder spacing.

The minimum amount of longitudinal reinforcement required for temperature and shrinkage in each of the top and bottom layers is given by **LRFD [5.10.8]** as follows:

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

and

$$0.11 \leq A_s \leq 0.60$$

Where:

A_s = Area of reinforcement in each direction and each face (in.^{2}/ft.)

f_y = Reinforcing steel yield strength = 60 ksi

b = Width of deck (inches)

h = Thickness of deck (inches)

In addition, the minimum amount of longitudinal steel in both layers used by WisDOT is #4 bars at 9" spacing to reduce transverse deck cracking. Identical amounts of steel are placed in both the top and bottom layer, and the reinforcing bars are uniformly spaced from edge to edge of slab. Table 17.5-3 and Table 17.5-4 use the same longitudinal bar spacings throughout a given bridge deck.

See Chapter 19 – Prestressed Concrete for design guidance regarding continuity reinforcement for prestressed girder bridges.

When continuous steel girders are not designed for negative composite action, **LRFD [6.10.1.7]** requires an area of longitudinal steel in both the top and bottom layer equal to 1% of the cross-sectional area of the slab in the span negative moment regions. The "d" value used for this computation is the total slab thickness excluding the wearing surface. This reinforcing steel is uniformly spaced from edge to edge of slab in the top and bottom layer. It is required that two-thirds of this reinforcement be placed in the top layer. The values shown in Table 17.5-3 and Table 17.5-4 provide adequate reinforcement to cover the requirements of **LRFD [6.10.1.7]**. It is WisDOT practice to abide by **LRFD [6.10.1.7]** for new bridges utilizing negative composite action, as well. See 24.7.6 for determining continuity bar cutoff locations for new bridges and rehabilitation bridges.

Longitudinal Reinforcing Steel for Deck Slabs on Girders for New Bridges and Deck Replacements, HL-93 Loading, Slab Thickness "T" ≥ 8"			
		Bar Size and Spacing (Inches)	
		Prestressed Girder Bridges	Steel Girder Bridges
Slab Thickness "T" (Inches)	Girder Spacing "S"	#4's Top and Bottom, Continuity Reinforcement To Be Designed (Top)	#4's Top and Bottom, Continuity Reinforcement** #6's (Top)
8	4'-6"	9.0	8.5
8	4'-9"	9.0	8.5

8	5'-0"	9.0	8.5
8	5'-3"	9.0	8.5
8	5'-6"	9.0	8.5
8	5'-9"	9.0	8.5
8	6'-0"	9.0	8.5
8	6'-3"	9.0	8.5
8	6'-6"	9.0	8.5
8	6'-9"	9.0	8.5
8	7'-0"	9.0	8.5
8	7'-3"	9.0	8.5
8	7'-6"	8.5	8.5
8	7'-9"	8.5	8.5
8	8'-0"	8.0	8.5
8.5	8'-3"	9.0	8.0
8.5	8'-6"	8.5	8.0
8.5	8'-9"	8.5	8.0
8.5	9'-0"	8.5	8.0
8.5	9'-3"	8.0	8.0
9	9'-6"	9.0	7.5
9	9'-9"	8.5	7.5
9	10'-0"	8.5	7.5
9	10'-3"	8.0	7.5
9	10'-6"	8.0	7.5
9.5	10'-9"	8.0	7.0
9.5	11'-0"	8.0	7.0
9.5	11'-3"	8.0	7.0
9.5	11'-6"	8.0	7.0
10	11'-9"	8.0	6.5
10	12'-0"	8.0	6.5
10	12'-3"	8.0	6.5
10	12'-6"	8.0	6.5
10.5	12'-9"	8.5	6.0
10.5	13'-0"	8.0	6.0
10.5	13'-3"	8.0	6.0
10.5	13'-6"	8.0	6.0
11	13'-9"	8.0	6.0
11	14'-0"	8.0	6.0

Legend:

** Use for deck slabs on steel girders in negative moment regions. New bridge shall be designed for composite action in the negative moment region.

Table 17.5-3

Longitudinal Reinforcing Steel For Deck Slabs on Girders
for New Bridges and Deck Replacements, HL-93 Loading, Slab Thickness “T” ≥ 8”

Longitudinal Reinforcing Steel for Deck Slabs on Girders for Deck Replacements, HL-93 Loading, Slab Thickness “T” < 8”			
Slab Thickness “T” (Inches)	Girder Spacing “S”	Bar Size and Spacing (Inches)	
		Prestressed Girder Bridges	Steel Girder Bridges
		#4’s Top and Bottom, Continuity Reinforcement To Be Designed (Top)	#4’s Top and Bottom, Continuity Reinforcement** #6’s (Top)
6.5	4’-0”	7.0	7.0
6.5	4’-3”	7.0	7.0
6.5	4’-6”	7.0	7.0
6.5	4’-9”	7.0	7.0
6.5	5’-0”	7.0	7.0
6.5	5’-3”	7.0	7.0
6.5	5’-6”	7.0	7.0
6.5	5’-9”	6.5	6.5
6.5	6’-0”	6.5	6.5
6.5	6’-3”	6.5	6.5
6.5	6’-6”	6.5	6.5
6.5	6’-9”	6.0	6.0
6.5	7’-0”	6.0	6.0
7	4’-0”	8.0	8.0
7	4’-3”	8.0	8.0
7	4’-6”	8.0	8.0
7	4’-9”	8.0	8.0
7	5’-0”	8.0	8.0
7	5’-3”	8.0	8.0
7	5’-6”	8.0	8.0
7	5’-9”	7.5	7.5
7	6’-0”	7.5	7.5
7	6’-3”	7.5	7.5
7	6’-6”	7.0	7.0
7	6’-9”	7.0	7.0
7	7’-0”	7.0	7.0

7	7'-3"	6.5	6.5
7	7'-6"	6.5	6.5
7	7'-9"	6.5	6.5
7	8'-0"	6.0	6.0
7.5	4'-0"	9.0	9.0
7.5	4'-3"	9.0	9.0
7.5	4'-6"	9.0	9.0
7.5	4'-9"	9.0	9.0
7.5	5'-0"	9.0	9.0
7.5	5'-3"	9.0	9.0
7.5	5'-6"	9.0	9.0
7.5	5'-9"	8.5	8.5
7.5	6'-0"	8.5	8.5
7.5	6'-3"	8.5	8.5
7.5	6'-6"	8.0	8.0
7.5	6'-9"	8.0	8.0
7.5	7'-0"	7.5	7.5
7.5	7'-3"	7.5	7.5
7.5	7'-6"	7.5	7.5
7.5	7'-9"	7.0	7.0
7.5	8'-0"	7.0	7.0
7.5	8'-3"	6.5	6.5
7.5	8'-6"	6.5	6.5
7.5	8'-9"	6.5	6.5
7.5	9'-0"	6.0	6.0
7.5	9'-3"	6.0	6.0
7.5	9'-6"	5.5	5.5

Legend:

** Use for deck slabs on steel girders in negative moment regions when not designed for negative moment composite action.

Table 17.5-4

Longitudinal Reinforcing Steel for Deck Slabs
on Girders for Deck Replacements, HL-93 Loading, Slab Thickness "T" < 8"
(Only use Table 17.5-4 if Bridge Rating is unacceptable with "T" ≥ 8")

The longitudinal reinforcing steel presented in Table 17.5-3 and Table 17.5-4 is designed in accordance with *AASHTO LRFD*. The tables are developed based on deck concrete with a 28-day compressive strength of f'_c = 4 ksi and reinforcing steel with a yield strength of f_y = 60 ksi. The dead load includes 20 psf for future wearing surface.

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 WisDOT 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.

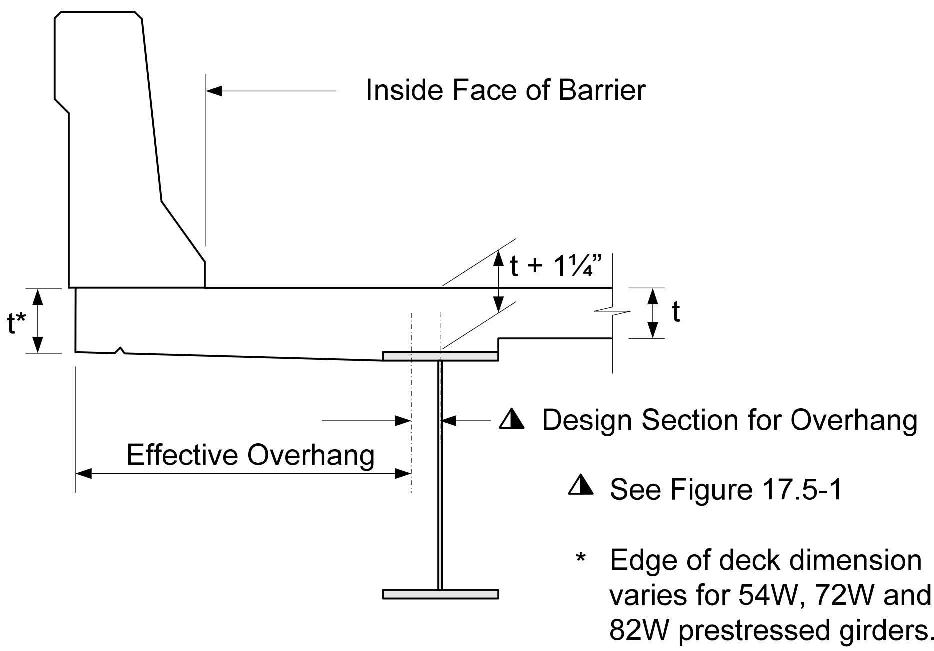


Figure 17.6-1
Deck Overhang Design Section

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

Lapping every other bar: use #4's @ 17", As = 0.14 in^2/ft, use Detail “A”.

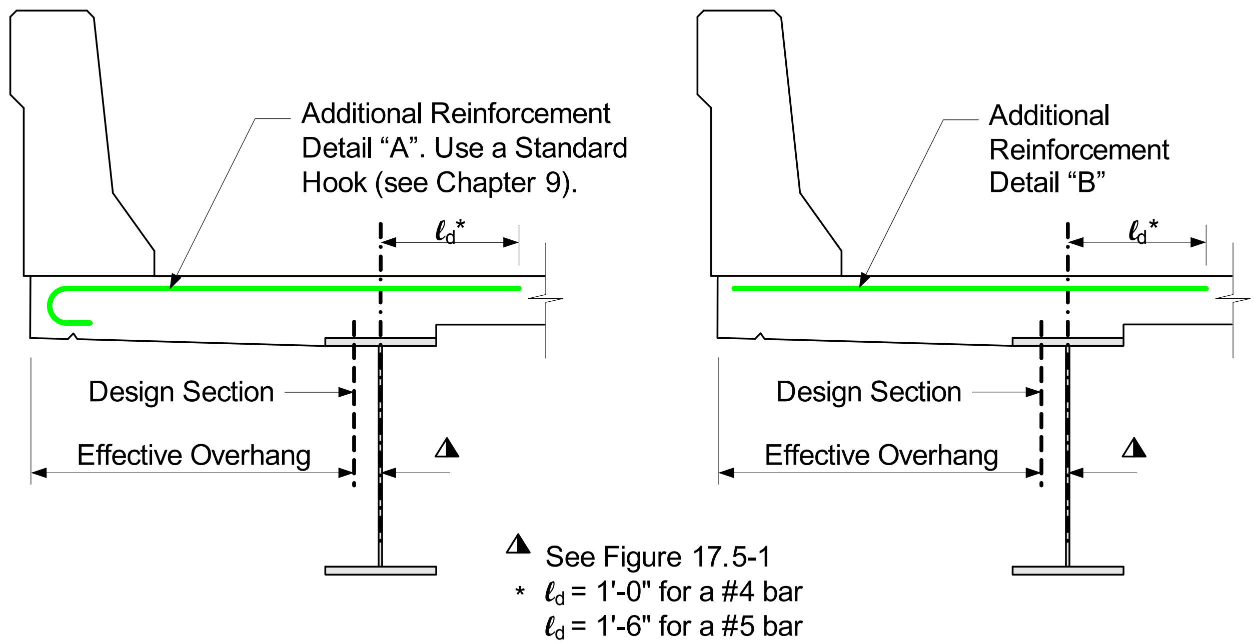


Figure 17.6-8
Overhang Reinforcement Details

To reiterate:

1. Details for additional overhang reinforcement are shown in Figure 17.6-8. Detail “A” shall be used with Table 17.6-2, Table 17.6-3 and Table 17.6-5. Detail “B” shall be used with Table 17.6-4.

2. Girder Type 1 shall include steel girders and the following prestressed girders; 28-inch, 36-inch, 36W-inch, 45W-inch. Prestressed girders used for rehabilitation projects, 45-inch, 54-inch and 70-inch, shall also be considered to be Girder Type 1. Girder Type 2 shall include the following prestressed girders; 54W-inch, 72W-inch and 82W-inch.

17.7 Construction Joints

Optional transverse construction joints are permitted on continuous concrete deck structures to limit the concrete volume in a single pour. Refer to the Standard Detail for Slab Pouring Sequence for the optimum slab pouring sequence. On structures over 300 feet long, transverse construction joints, if used, are to be placed at 0.6 of the span length beyond the pier in the direction of the pour. For continuous prestressed concrete girder bridges, optional transverse construction joints should be located midway between the cut-off points for continuity reinforcing steel or at 0.75 of the span, whichever is closest to the pier.

The rate of placing concrete for continuous steel girders shall equal or exceed 0.5 of the span length per hour but need not exceed 100 cubic yards per hour. Transverse construction joints may be omitted with approval of Bureau of Structures.

Optional longitudinal construction joints, if used, are to be approved by the engineer and preferably located beneath the median or parapet. Otherwise, the joint should be located along the edge of the lane line. When the width of a superstructure exceeds 90 feet, a longitudinal construction joint with reinforcement through the joint shall be detailed. Longitudinal joints should also be at least 6 inches from the edge of the top flange of the girder. Open joints may be used in a median or between parapets. Consideration should be given to sealing open joints with compression seals or other sealants. A longitudinal construction joint detail is provided in the Standard Details.

The structure plans should permit the contractor to propose an alternate construction joint schedule subject to approval of the engineer.

Table of Contents

18.1 Introduction

18.1.1 General

This chapter considers the following types of concrete structures:

- Flat Slab
- Haunched Slab

A longitudinal slab is one of the least complex types of bridge superstructures. It is composed of a single element superstructure in comparison to the two elements of the transverse slab on girders or the three elements of a longitudinal slab on floor beams supported by girders. Due to simplicity of design and construction, the concrete slab structure is relatively economical. Its limitation lies in the practical range of span lengths and maximum skews for its application. For longer span applications, the dead load becomes too high for continued economy. Application of the haunched slab has increased the practical range of span lengths for concrete slab structures.

18.1.2 Limitations

Concrete slab structure types are not recommended over streams where the normal water freeboard is less than 4 feet; formwork removal requires this clearance. When spans exceed 35 feet, freeboard shall be increased to 5 feet above normal water.

All concrete slab structures are limited to a maximum skew of 30 degrees. Slab structures with skews in excess of 30 degrees, require analysis of complex boundary conditions that exceed the capabilities of the present design approach used in the Bureau of Structures.

Continuous span slabs are to be designed using the following pier types:

- Piers with pier caps (on columns or shafts)
- Wall type piers

These types will allow for ease of future superstructure replacement. Piers that have columns without pier caps, have had the columns damaged during superstructure removal. This type of pier will not be allowed without the approval of the Structures Design Section.

18.2 Specifications, Material Properties and Structure Type

18.2.1 Specifications

Reference may be made to the design and construction related material as presented in the following specifications:

- *State of Wisconsin, Department of Transportation Standard Specifications for Highway and Structure Construction*

  Section 502 - Concrete Bridges

  Section 505 - Steel Reinforcement

- Other Specifications as referenced in Chapter 3

18.2.2 Material Properties

The properties of materials used for concrete slab structures are as follows:

f'_c = specified compressive strength of concrete at 28 days, based on cylinder tests

4 ksi, for concrete slab superstructure

3.5 ksi, for concrete substructure units

f_y = 60 ksi, specified minimum yield strength of reinforcement (Grade 60)

E_s = 29,000 ksi, modulus of elasticity of steel reinforcement **LRFD [5.4.3.2]**

E_c = modulus of elasticity of concrete in slab **LRFD [5.4.2.4]**

= $33{,}000\ K_1\ w_c^{1.5}\ (f'_c)^{1/2} = 3800$ ksi

Where:

K_1 = 1.0

w_c = 0.150 kcf, unit weight of concrete

n = $E_s / E_c = 8$ **LRFD [5.7.1]** (modular ratio)

18.2.3 Structure Type and Slab Depth

Prepare preliminary structure data, looking at the type of structure, span lengths, approximate slab depth, skew, roadway width, etc.. The selection of the type of concrete slab

Where:

E = equivalent distribution width (ft)

Look at the distribution factor (for axle loads) calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

Compute the distribution factor associated with tributary portion of design lane load, to be applied to <u>full lane load</u>: **LRFD [3.6.1.2.4]**

$$DF = \frac{\left[\dfrac{(\text{SWL})}{(10\,\text{ft lane load width})}\right]}{(\text{E})}$$

Where:

E = equivalent distribution width (ft)

SWL = slab width loaded (ft) $\geq$ 0.

E – (distance from edge of slab to inside face of barrier or sidewalk)

Look at the distribution factor (for lane load) calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

18.4.6 Longitudinal Slab Reinforcement

The concrete cover on the top bars is 2 ½ inches, which includes a ½ inch wearing surface. The bottom bar cover is 1 ½ inches. Minimum clear spacing between adjacent longitudinal bars is 3 ½ inches. The maximum center to center spacing of adjacent bars shall not exceed 1.5 times the thickness of the slab or 18.0 inches **LRFD [5.10.3.2]**. When bundled bars are used, see **LRFD [5.10.3.1.5, 5.11.2.3, 5.11.5.2.1]**.

18.4.6.1 Design for Strength

Strength Limit State considerations and assumptions are detailed in **LRFD [5.5.4, 5.7.2]**.

The area of longitudinal slab reinforcement, A_s , should be designed for strength at maximum moment locations along the structure, and for haunched slab structures, checked for strength at the haunch/slab intercepts. The area should also be checked for strength at bar reinforcement cutoff locations. This reinforcement should be designed for interior and exterior strips (edge beams) in both positive and negative moment regions. The reinforcement in the exterior strip is always equal to or greater than that required for the slab in an interior strip. Compare the reinforcement to be used for each exterior strip and select the strip with the largest amount of reinforcement (in^2/ft). Use this reinforcement pattern for both exterior strips to keep the bar layout symmetrical. Concrete parapets, curbs, sidewalks and other appurtenances are not to be considered to provide strength to the edge beam **LRFD [9.5.1]**. The total factored moment, M_u , shall be calculated using factored loads described in 18.3.3.1 for Strength I Limit State. Then calculate the coefficient of resistance, R_u :

$$R_u = M_u / \phi \, b \, d_s^2$$

Where:

ϕ	=	0.90 (see 18.3.3.2)
b	=	12 in (for a 1 foot design slab width)
d_s	=	slab depth (excl. ½ inch wearing surface) – bar clearance – ½ bar diameter (in)

Calculate the reinforcement ratio, ρ, using (R_u vs. ρ) Table 18.4-3 .

Then calculate required area,

$$A_s = \rho \, (b) \, (d_s)$$

Area of bar reinforcement per foot of slab width can be found in Table 18.4-4 .

The factored resistance, M_r , or moment capacity, shall be calculated as in 18.3.3.2.1.

Then check that, $M_u \leq M_r$ is satisfied.

The area of longitudinal reinforcement, A_s , should also be checked for moment capacity (factored resistance) along the structure, to make sure it can handle factored moments due to applied dead load (including future wearing surface) and the Wisconsin Standard Permit Vehicle (Wis-SPV) (with a minimum gross vehicle load of 190 kips) on an interior strip. This requirement is stated in 17.1.2.1. See Chapter 45 for details on checking the capacity of the structure for this Permit Vehicle.

18.4.6.2 Check for Fatigue

Fatigue Limit State considerations and assumptions are detailed in **LRFD [5.5.3, 5.7.1, 9.5.3]**

The area of longitudinal slab reinforcement, A_s , should be checked for fatigue stress range at locations where maximum stress range occurs along the structure, and for haunched slab structures, checked at the haunch/slab intercepts. The area should also be checked for fatigue stress range at bar reinforcement cutoff locations using Fatigue I Limit State. Check the reinforcement in an interior strip, where the largest number of fatigue cycles will occur.

Fatigue life of reinforcement is reduced by increasing the maximum stress level, bending of the bars and splicing of reinforcing bars by welding.

E18-1 Continuous 3-Span Haunched Slab - LRFD

A continuous 3-span haunched slab structure is used for the design example. The same basic procedure is applicable to continuous flat slabs. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. Design using a slab width equal to one foot. ***(Example is current through LRFD Fifth Edition - 2010 Interim)***

E18-1.1 Structure Preliminary Data

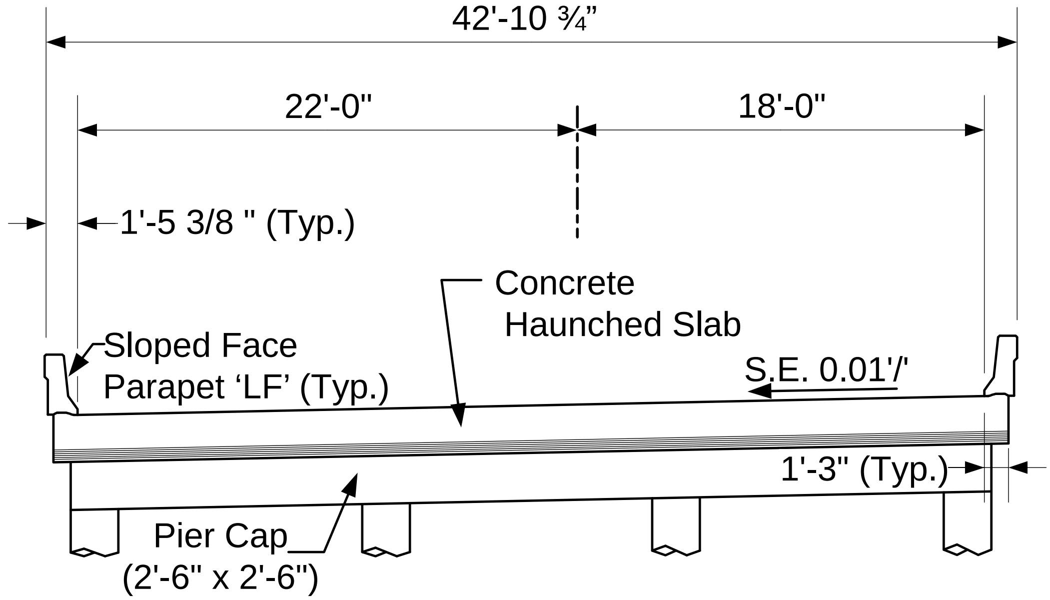


Figure E18.1

Section Perpendicular to Centerline

Live Load: HL-93
(A1) Fixed Abutments at both ends
Parapets placed after falsework is released

Geometry:

$L_1 := 38.0$	ft	Span 1
$L_2 := 51.0$	ft	Span 2
$L_3 := 38.0$	ft	Span 3
$slab_{width} := 42.5$	ft	out to out width of slab
$skew := 6$	deg	skew angle (RHF)
$w_{roadway} := 40.0$	ft	clear roadway width

Material Properties: (See 18.2.2)

$f'_c := 4$	ksi	concrete compressive strength

$f_y := 60$ ksi yield strength of reinforcement

$E_c := 3800$ ksi modulus of elasticity of concrete

$E_s := 29000$ ksi modulus of elasticity of reinforcement

$n := 8$ E_s / E_c (modular ratio)

Weights:

$w_c := 150$ pcf concrete unit weight

$w_{LF} := 387$ plf weight of Type LF parapet (each)

E18-1.2 LRFD Requirements

For concrete slab design, the slab dimensions and the size and spacing of reinforcement shall be selected to satisfy the equation below for all appropriate Limit States: (See 18.3.2.1)

$$Q = \Sigma\eta_i \cdot \gamma_i \cdot Q_i \le \phi \cdot R_n = R_r \qquad \text{(Limit States Equation)}$$

The value of the load modifier is:

$\eta_i := 1.0$ for all Limit States (See 18.3.2.2)

The force effect, Q_i , is the moment, shear, stress range or deformation caused by applied loads.

The applied loads from **LRFD [3.3.2]** are:

DC = dead load of slab (DC_{slab}), ½ inch wearing surface ($DC_{1/2"WS}$) and parapet dead load (DC_{para}) - (See E18-1.3)

DW = dead load of future wearing surface (DW_{FWS}) - (See E18-1.3)

LL+IM = vehicular live load (LL) with dynamic load allowance (IM) - (See E18-1.4)

The Influence of ADTT and skew on force effects, Q_i , are ignored for slab bridges (See 18.3.2.2).

The values for the load factors, γ_i , (for each applied load) and the resistance factors, ϕ , are found in Table E18.1.

The total factored force effect, Q , must not exceed the factored resistance, R_r . The nominal resistance, R_n , is the resistance of a component to the force effects.

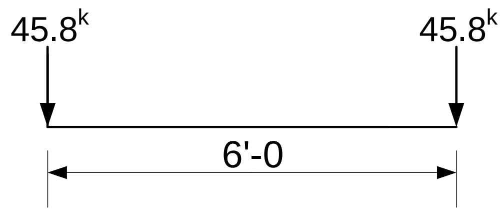


Figure E18.16

Design Truck Reaction

Design Lane Load Reaction (IM not applied to Lane Load):

$$\frac{(35.1)\,kip}{(10)\_ft\,lane} = 3.51\,\frac{kip}{ft}$$

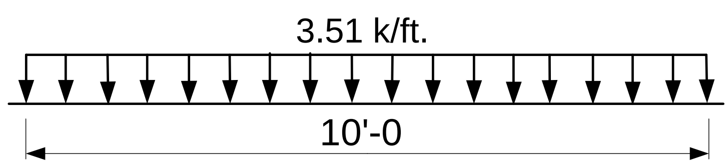


Figure E18.17

Design Lane Load Reaction

This live load is carried by the pier cap and a transverse beam represented by a portion of the slab over the pier.

Using influence lines for a 3-span continuous beam, the following results are obtained. The multiple presence factor (m) is 1.0 for (2) loaded lanes. **LRFD [3.6.1.1.2]**.

Calculate the positive live load moment, M_{LL+IM} , at (0.4 pt.) of Exterior Span

Because lane width of (10 ft) is almost equal to the span length (13.07 ft), for simplicity place uniform lane load reaction across the entire span, as shown in Figure E18.18.

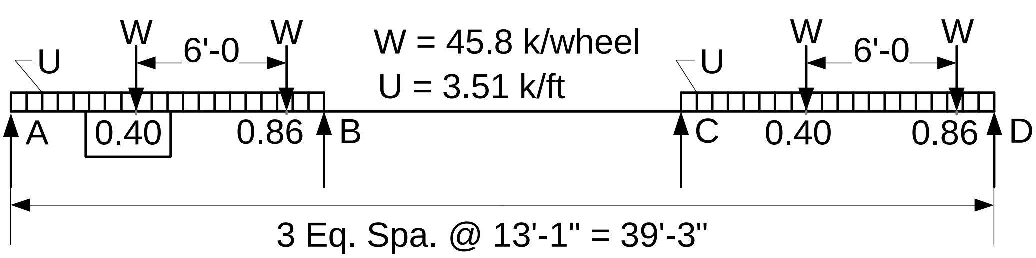


Figure E18.18

Live Load Placement for $+M_{LL+IM}$

M_{LL+IM} = (0.2042 + 0.0328 + 0.0102 + 0.0036)(45.8)(13.07) + (0.100)(3.51)(13.07)^2
= 150.1 + 60.0
= 210.1 kip-ft (Max + M_{LL+IM} in Ext. Span - 0.4 pt.)

Calculate the negative live load moment, $\underline{M_{LL+IM}}$, at C/L of column B

Because lane width of (10 ft) is almost equal to the span length (13.07 ft), for simplicity place uniform lane load reaction across the entire span, as shown in Figure E18.19.

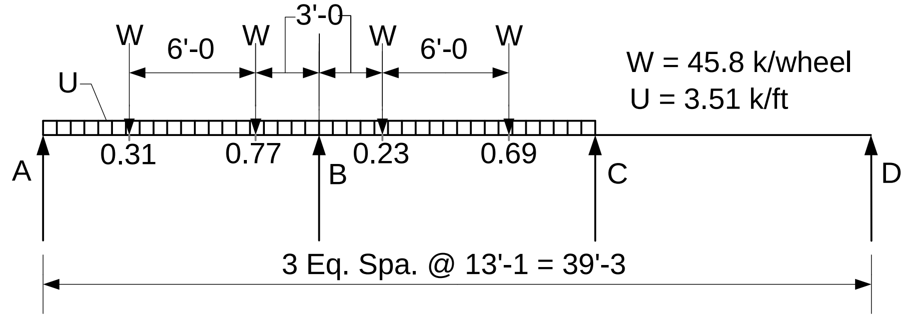


Figure E18.19

Live Load Placement for -M_{LL+IM}

M_{LL+IM} = (0.07448 + 0.08232 + 0.0679 + 0.0505)(45.8)(13.07) + (0.1167)(3.51)(13.07)^2
= 164.7 + 70.0
= 234.7 kip-ft (Max - M_{LL+IM} at C/L of column B)

It is assumed for this example that adequate shear transfer has been achieved **LRFD [5.8.4]** between transverse slab member and pier cap and that they will perform as a unit. Therefore, "FWS + para (DL)" and "LL + IM" will be acting on a member made up of the pier cap and the transverse slab member. Designer must insure adequate transfer if using this approach.

Calculate section width, b_{pos}, and effective depth, d_{pos} , in positive moment region, for the pier cap and the transverse slab member acting as a unit (See Figure E18.20):

b_{pos} = width of slab section = 1/2 center to center column spacing or 8 feet, whichever is smaller (See 18.4.7.2).

(C/L - C/L) column spacing x (1/2) = 6.5 ft < 8.0 ft $\boxed{b_{pos} = 78}$ in

d_{pos} = D_{haunch} + cap depth - bott. clr. - stirrup dia. - 1/2 bar dia.

$d_{pos} := 28 + 30 - 1.5 - 0.625 - 0.44$ $\boxed{d_{pos} = 55.44}$ in

Calculate section width, b_{neg}, and effective depth, d_{neg} , in negative moment region, for the pier cap and the transverse slab member acting as a unit (See Figure E18.20):

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19.3 Pretensioned Member Design

This section outlines several important considerations associated with the design of conventional pretensioned members.

19.3.1 Design Strengths

The typical specified design strengths for pretensioned members are:

Prestressed I-girder concrete:	f'_c	= 6 to 8 ksi
Prestressed box girder concrete:	f'_c	= 5 ksi
Prestressed concrete (at release):	f'_{ci}	= 0.75 to 0.85 $f'_c \leq 6.8$
Deck and diaphragm concrete:	f'_c	= 4 ksi
Prestressing steel:	f_{pu}	= 270 ksi
Grade 60 reinforcement:	f_y	= 60 ksi

The *actual required* compressive strength of the concrete at prestress transfer, f'_{ci}, is to be stated on the plans. For typical prestressed girders, $f'_{ci(min)}$ is $0.75(f'_c)$.

WisDOT policy item:

The use of concrete with strength greater than 8 ksi is only allowed with the prior approval of the BOS Development Section. Occasional use of strengths up to 8.5 ksi may be allowed. Strengths exceeding these values are difficult for local fabricators to consistently achieve as the coarse aggregate strength becomes the controlling factor.

The use of 8 ksi concrete for I-girders and 6.8 ksi for f'_{ci} still allows the fabricator to use a 24-hour cycle for girder fabrication. There are situations in which higher strength concrete in the I-girders may be considered for economy, provided that f'_{ci} does not exceed 6.8 ksi. Higher strength concrete may be considered if the extra strength is needed to avoid using a less economical superstructure type or if a shallower girder can be provided and its use justified for sufficient reasons (min. vert. clearance, etc.) Using higher strength concrete to eliminate a girder line is not the preference of the Bureau of Structures. It is often more economical to add an extra girder line than to use debonded strands with the minimum number of girder lines. After the number of girders has been determined, adjustments in girder spacing should be investigated to see if slab thickness can be minimized and balance between interior and exterior girders optimized.

Prestressed I-girders below the required 28-day concrete strength (or 56-day concrete strength for f'_c = 8 ksi) will be accepted if they provide strength greater than required by the design and at the reduction in pay schedule in the *Wisconsin Standard Specifications for Highway and Structure Construction*.

Low relaxation prestressing strands are required.

19.3.2 Loading Stages

The loads that a member is subjected to during its design life and those stages that generally influence the design are discussed in **LRFD [5.9]** and in the following sections. The allowable stresses at different loading stages are defined in **LRFD [5.9.3]** and **LRFD [5.9.4]**.

19.3.2.1 Prestress Transfer

Prestress transfer is the initial condition of prestress that exists immediately following the release of the tendons (transfer of the tendon force to the concrete). The eccentricity of the prestress force produces an upward camber. In addition, a stress due to the dead load of the member itself is also induced. This is a stage of temporary stress that includes a reduction in prestress due to elastic shortening of the member.

19.3.2.2 Losses

After elastic shortening losses, the external loading is the same as at prestress transfer. However, the internal stress due to the prestressing force is further reduced by losses resulting from relaxation due to creep of the prestressing steel together with creep and shrinkage of the concrete. It is assumed that all losses occur prior to application of service loading.

LRFD [5.9.5] provides guidance about prestress losses for both pretensioned and post-tensioned members. This section presents a refined and approximate method for the calculation of time-dependent prestress losses such as concrete creep and shrinkage and prestressing steel relaxation.

<u>WisDOT policy item:</u>

WisDOT policy is to use the approximate method described in **LRFD [5.9.5.3]** to determine time-dependent losses, since this method does not require the designer to assume the age of the concrete at the different loading stages.

Losses for pretensioned members that are considered during design are listed in the following sections.

19.3.2.2.1 Elastic Shortening

Per **LRFD [5.9.5.2.3a]**, the loss due to elastic shortening, Δf_{pES1} (ksi), in pretensioned concrete members shall be taken as:

$$\Delta f_{pES1} = \frac{E_p}{E_{ct}} f_{cgp}$$

Where:

E_p = Modulus of elasticity of prestressing steel = 28,500 ksi **LRFD**

[5.4.4.2]

E_{ct} = Modulus of elasticity of concrete at transfer or time of load application in ksi (see 19.3.3.8)

f_{gcp} = Concrete stress at the center of gravity of prestressing tendons due to the prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment (ksi)

19.3.2.2.2 Time-Dependent Losses

Per **LRFD [5.9.5.3],** an estimate of the long-term losses due to steel relaxation as well as concrete creep and shrinkage on standard precast, pretensioned members shall be taken as:

$$\Delta f_{pLT} = 10.0\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 12.0\gamma_h\gamma_{st} + \Delta f_{pR}$$

Where:

$$\gamma_h = 1.7 - 0.01H$$

$$\gamma_{st} = \frac{5}{(1 + f'_{ci})}$$

f_{pi} = Prestressing steel stress immediately prior to transfer (ksi)

H = Average annual ambient relative humidity in %, taken as 72% in Wisconsin

Δf_{pR} = Relaxation loss estimate taken as 2.5 ksi for low relaxation strands or 10.0 ksi for stress-relieved strands (ksi)

The losses due to elastic shortening must then be added to these time-dependent losses to determine the total losses. For members made without composite deck slabs such as box girders, time-dependent losses shall be determined using the refined method of **LRFD [5.9.5.4]**. For non-standard members with unusual dimensions or built using staged segmental construction, the refined method of **LRFD [5.9.5.4]** shall also be used.

19.3.2.2.3 Fabrication Losses

Fabrication losses are not considered by the designer, but they affect the design criteria used during design. Anchorage losses which occur during stressing and seating of the prestressed strands vary between 1% and 4%. Losses due to temperature change in the strands during cold weather prestressing are 6% for a 60°F change. The construction specifications permit a 5% difference in the jack pressure and elongation measurement without any adjustment.

19.3.2.3 Service Load

During service load, the member is subjected to the same loads that are present after prestress transfer and losses occur, in addition to the effects of the I-girder and box girder load-carrying behavior described in the next two sections.

19.3.2.3.1 I-Girder

In the case of an I-girder, the dead load of the deck and diaphragms are always carried by the basic girder section on a simple span. At strand release, the girder dead load moments are calculated based on the full girder length. For all other loading stages, the girder dead load moments are based on the span length. This is due to the type of construction used (that is, nonshored girders simply spanning from one substructure unit to another for single-span as well as multi-span structures).

The live load plus dynamic load allowance along with any superimposed dead load (curb, parapet or median strip which is placed after the deck concrete has hardened) are carried by the continuous composite section.

WisDOT exception to AASHTO:

The standard pier diaphragm is considered to satisfy the requirements of **LRFD [5.14.1.4.5]** and shall be considered to be fully effective.

In the case of multi-span structures with fully effective diaphragms, the longitudinal distribution of the live load, dynamic load allowance and superimposed dead loads are based on a continuous span structure. This continuity is achieved by:

a. Placing non-prestressed (conventional) reinforcement in the deck area over the interior supports.

b. Casting concrete between and around the abutting ends of adjacent girders to form a diaphragm at the support. Girders shall be in line at interior supports and equal numbers of girders shall be used in adjacent spans. The use of variable numbers of girders between spans requires prior approval by BOS.

If the span length ratio of two adjacent spans exceeds 1.5, the girders are designed as simple spans. In either case, the stirrup spacing is detailed the same as for continuous spans and bar steel is placed over the supports equivalent to continuous span design. It should be noted that this value of 1.5 is not an absolute structural limit.

19.3.2.3.2 Box Girder

In the case of slabs and box girders with a bituminous or thin concrete surface, the dead load together with the live load and dynamic load allowance are carried by the basic girder section.

When this girder type has a concrete floor, the dead load of the floor is carried by the basic section and the live load, dynamic load allowance and any superimposed dead loads are

$$\Delta_s = \frac{M_1L^2}{8E_iI_b}\left(\frac{12}{1}\right)\left(\frac{12^2}{1}\right) = \frac{M_1L^2}{8E_iI_b}\left(\frac{1728}{1}\right)$$

$$\Delta_s = \frac{216M_1L^2}{E_iI_b}$$ (with units as shown below)

Where:

Δ_s	=	Deflection due to force in the straight strands minus elastic shortening loss (in)
L	=	Span length between centerlines of bearing (ft)
E_i	=	Modulus of elasticity at the time of release (see 19.3.3.8) (ksi)
I_b	=	Moment of inertia of basic beam (in^4)

The draped strands induce the following moments at the ends and within the span:

$M_2 = \frac{1}{12}\left(P_i^D\left(A - C\right)\right)$, which produces upward deflection, and

$M_3 = \frac{1}{12}\left(P_i^D\left(A - y_B\right)\right)$, which produces downward deflection when A is greater than y_B

Where:

M_2, M_3	=	Components of moment due to initial prestress force in the draped strands minus the elastic shortening loss (k-ft)
P_i^D	=	Initial prestress force in the draped strands minus the elastic shortening loss (kips)
A	=	Distance from bottom of beam to center of gravity of draped strands at centerline of bearing (in)
C	=	Distance from bottom of beam to center of gravity of draped strands between hold-down points (in)

These moments produce a net upward deflection at midspan, which is given by:

$$\Delta_D = \frac{216L^2}{E_iI_b}\left(\frac{23}{27}M_2 - M_3\right)$$

Where:

Δ_D	=	Deflection due to force in the draped strands minus elastic shortening loss (in)

The combined upward deflection due to prestress is:

$$\Delta_{PS} = \Delta_s + \Delta_D = \frac{216L^2}{E_i I_b}\left(M_1 + \frac{23}{27}M_2 - M_3\right)$$

Where:

Δ_{PS} = Deflection due to straight and draped strands (in)

The downward deflection due to beam self-weight at release is:

$$\Delta_{o(DL)} = \frac{5W_b L^4}{384E_i I_b} \quad \text{(with all units in inches and kips)}$$

Using unit weights in kip per foot, span lengths in feet, E in ksi and I_b in inches^4, this equation becomes the following:

$$\Delta_s = \frac{5W_b L^4}{384E_i I_b}\left(\frac{1}{12}\right)\left(\frac{12^4}{1}\right) = \frac{5W_b L^4}{384E_i I_b}\left(\frac{20736}{12}\right)$$

$$\Delta_{o(DL)} = \frac{22.5W_b L^4}{E_i I_b} \quad \text{(with units as shown below)}$$

Where:

$\Delta_{o(DL)}$ = Deflection due to beam self-weight at release (in)

W_b = Beam weight per unit length (k/ft)

Therefore, the anticipated prestress camber at release is given by:

$$\Delta_i = \Delta_{PS} - \Delta_{o(DL)}$$

Where:

Δ_i = Prestress camber at release (in)

Camber, however, continues to grow after the initial strand release. For determining substructure beam seats, average concrete haunch values (used for both DL and quantity calculations) and the required projection of the vertical reinforcement from the tops of the prestressed girders, **a camber multiplier of 1.4 shall be used**. This value is multiplied by the theoretical camber at release value.

19.3.3.18.2 Dead Load Deflection

The downward deflection due to the dead load of the deck and midspan diaphragm is:

$$\Delta_{nc\,(DL)} = \frac{5W_{deck}L^4}{384EI_b} + \frac{P_{dia}L^3}{48EI_b} \quad \text{(with all units in inches and kips)}$$

Using span lengths in units of feet, unit weights in kips per foot, E in ksi, and I_b in inches^4, this equation becomes the following:

$$\Delta_s = \frac{5W_{deck}L^4}{384EI_b}\left(\frac{1}{12}\right)\left(\frac{12^4}{1}\right) + \frac{P_{dia}L^3}{48EI_b}\left(\frac{12^3}{1}\right) = \frac{5W_{deck}L^4}{384EI_b}\left(\frac{20736}{12}\right) + \frac{P_{dia}L^3}{48EI_b}\left(\frac{1728}{1}\right)$$

$$\Delta_{o(DL)} = \frac{22.5W_bL^4}{EI_b} + \frac{36P_{dia}L^3}{EI_b} \quad \text{(with units as shown below)}$$

Where:

$\Delta_{nc\,(DL)}$	=	Deflection due to non-composite dead load (deck and midspan diaphragm) (in)
W_{deck}	=	Deck weight per unit length (k/ft)
P_{dia}	=	Midspan diaphragm weight (kips)
E	=	Girder modulus of elasticity at final condition (see 19.3.3.8) (ksi)

A similar calculation is done for parapet and sidewalk loads on the composite section. Provisions for deflections due to future wearing surface shall not be included.

For girder structures with raised sidewalks, loads shall be distributed as specified in Chapter 17, and separate deflection calculations shall be performed for the interior and exterior girders.

19.3.3.18.3 Residual Camber

Residual camber is the camber that remains after the prestress camber has been reduced by the composite and non-composite dead load deflection. Residual camber is computed as follows:

$$RC = \Delta_i - \Delta_{nc(DL)} - \Delta_{c(DL)}$$

19.3.4 Deck Forming

Deck forming requires computing the relationship between the top of girder and bottom of deck necessary to achieve the desired vertical roadway alignment. Current practice for design is to use a minimum haunch of 2" at the edge of the girder flange. This haunch value is also used for calculating composite section properties. This will facilitate current deck forming practices which use 1/2" removable hangers and 3/4" plywood, and it will allow for variations in prestress camber. Also, future deck removal will be less likely to damage the top girder flanges. An average haunch height of 3 inches minimum can be used for determining haunch weight for preliminary design. It should be noted that the actual haunch values should be compared with the estimated values during final design. If there are significant differences in these values, the design should be revised. The actual average haunch height should be used to calculate the concrete quantity reported on the plans as well as the value reported on the prestressed girder details sheet. The actual haunch values at the girder ends shall be used for determining beam seat elevations.

For designs involving vertical curves, Figure 19.3-6 shows two different cases.

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36.1 Design Method

36.1.1 Design Requirements

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

36.1.2 Rating Requirements

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

WisDOT Policy Item:

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

36.1.3 Standard Permit Design Check

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

36.2 General

Box culverts are reinforced concrete closed rigid frames which must support vertical earth and truck loads and lateral earth pressure. They may be either single or multi-cell. The most common usage is to carry water under roadways, but they are frequently used for pedestrian or cattle underpasses.

The minimum size for pedestrian underpasses is 8 feet high by 5 feet wide. The minimum size for cattle underpasses is 6 feet high by 5 feet wide. A minimum vertical opening of 5 feet is desirable for concrete box culverts for cleaning purposes.

Aluminum box culverts are not permitted by the Bureau of Structures.

Typical sections for the most frequently used box culverts are shown below.

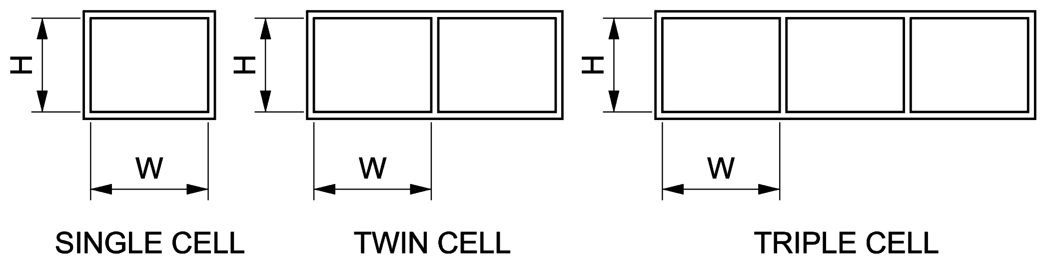


Figure 36.2-1
Typical Cross Sections

Hydraulic and other requirements at the site determine the required height and area of the box. Hydraulic design of box culverts is described in Chapter 8. Once the required height and area is determined, the selection of a single or multi-cell box is determined entirely from economics. Barrel lengths are computed to the nearest 6 inches. For multi-cell culverts the cell widths are kept equal.

36.2.1 Material Properties

The properties of materials used for concrete box culverts are as follows::

f'_c = specified compressive strength of concrete at 28 days, based on cylinder tests

= 3.5 ksi for concrete in box culverts

f_y = 60 ksi, specified minimum yield strength of reinforcement (Grade 60)

E_s = 29,000 ksi, modulus of elasticity of steel reinforcement **LRFD [5.4.3.2]**

E_c = modulus of elasticity of concrete in box **LRFD [5.4.2.4]**
= $(33{,}000)(K_1)(w_C)^{1.5}(f'_C)^{1/2}$ = 3586 ksi

Where:

K_1 = 1.0

W_C = 0.15 kcf, unit weight of concrete

n = Es / Ec = 8, modular ratio **LRFD [5.7.1]**

36.2.2 Bridge or Culvert

Occasionally, the waterway opening(s) for a highway-stream crossing can be provided for by either culvert(s) or bridge(s). Consider the hydraulics of the highway-stream crossing system in choosing the preferred design from the available alternatives. Estimates of life cycle costs and risks associated with each alternative help indicate which structure to select. Consider construction costs, maintenance costs, and risks of future costs to repair flood damage. Other considerations which may influence structure-type selection are listed in Table 36.2-1.

Bridges	
Advantages	Disadvantages
Less susceptible to clogging with drift, ice and debris	Require more structural maintenance than culverts
Waterway width increases with rising water surface until water begins to submerge structure	Piers and abutments susceptible to scour failure
Natural bottom for waterway	Susceptible to ice and frost forming on deck
Culverts	
Grade rises and widening projects sometimes can be accommodated by extending culvert ends	Silting in multiple barrel culverts may require periodic cleanout
Minimum structural maintenance	No increase in waterway area as stage rises above top of culvert
Usually easier and quicker to build than bridges	May clog with drift, debris or ice

Table 36.2-1
Advantages/Disadvantages of Structure Type

36.2.3 Staged Construction for Box Culverts

The inconvenience to the traveling public has often led to staged construction projects. Box culverts typically work well with staged construction. Any cell joint can be used for a staging joint. When the construction staging line cannot be determined in design to locate a cell joint, a contractor placed construction joint can be done with an extra set of dowel bars and the contractor field cutting the longitudinal bars.

36.3 Limit States Design Method

36.3.1 LRFD Requirements

For box culvert design, the component dimensions and the size and spacing of reinforcement shall be selected to satisfy the following equation for all appropriate limit states, as presented in **LRFD [1.3.2.1]:**

$$Q = \sum \eta_i \gamma_i Q_i \leq \phi R_n = R_r$$

Where:

η_i	=	Load modifier (a function of η_D, η_R, and η_i)
γ_i	=	Load factor
Q_i	=	Force effect: moment, shear, stress range or deformation caused by applied loads
Q	=	Total factored force effect
ϕ	=	Resistance factor
R_n	=	Nominal resistance: resistance of a component to force effects
R_r	=	Factored resistance = ϕR_n

See 17.2.2 for load modifier values.

36.3.2 Limit States

The Strength I Limit State is used to design reinforcement for flexure and checking shear in the slabs and walls, **LRFD [12.5.3]**. The Service I Limit State is used for checking reinforcement for crack control criteria, **LRFD [12.5.2]**, and checking settlement of the box culvert as shown in 36.8.1.

Per **LRFD [C12.5.3, 5.5.3],** buried structures have been shown not to be controlled by fatigue.

WisDOT Policy Item:

Fatigue criteria are not required on any reinforced concrete box culverts, with or without fill on the top slab of the culvert. This policy item is based on the technical paper titled "Fatigue Evaluation for Reinforced Concrete Box Culverts" by H Hany Maximos, Ece Erdogmus, and Maher Tadros, published in the ACI Structural Journal, January/February 2010.

36.3.3 Load Factors

In accordance with **LRFD [Table 3.4.1-1 and Table 3.4.1-2]**, the following Strength I load factors, γ_{st}, and Service I load factors, γ_{s1}, shall be used for box culvert design:

Type of Load		Strength I Load Factor, γ_{st} Max.	Min.	Service I Load Factor, γ_{s1}
Dead Load-Components	DC	1.25	0.90	1.0
Dead Load-Wearing Surface	DW	1.50	0.65	1.0
Vertical Earth Pressure	EV	1.35	0.90	1.0
Horizontal Earth Pressure	EH	1.50	0.50[1]	1.0
Live Load Surcharge	LS	1.75	1.75	1.0
Live Load + IM	LL+IM	1.75	1.75	1.0

[1]Per **LRFD [3.11.7]**, for culverts where earth pressure may reduce effects caused by other loads, a 50% reduction may be used, but not combined with the minimum load factor specified in **LRFD [Table 3.4.1-2]**.

36.3.4 Strength Limit State

Strength I Limit State shall be applied to ensure that strength and stability are provided to resist the significant load combinations that a structure is expected to experience during its design life **LRFD [1.3.2.4]**.

36.3.4.1 Factored Resistance

The resistance factor, ϕ, is used to reduce the computed nominal resistance of a structural element. This factor accounts for the variability of material properties, structural dimensions and workmanship, and uncertainty in prediction of resistance.

The resistance factors, ϕ, for reinforced concrete box culverts for the Strength Limit State per **LRFD [Table 12.5.5-1]** are as shown below:

Structure Type	Flexure	Shear
Cast-In-Place	0.90	0.85
Precast	1.00	0.90
Three-Sided	0.95	0.90

36.3.4.2 Moment Capacity

For rectangular sections, the nominal moment resistance, M_n, per **LRFD [5.7.3.2.3]** (tension reinforcement only) equals:

$$M_n = A_s f_s \left(d_s - \frac{a}{2}\right)$$

The factored resistance, M_r, or moment capacity per **LRFD [5.7.3.2.1]**, shall be taken as:

$$M_r = \phi M_n = \phi A_s f_s \left(d_s - \frac{a}{2}\right)$$

For additional information on concrete moment capacity, including stress and strain assumptions used, refer to 18.3.3.2.1.

The location of the design moment will consider the haunch dimensions in accordance with **LRFD [12.11.4.2]**. No portion of the haunch shall be considered in adding to the effective depth of the section.

36.3.4.3 Shear Capacity

Per **LRFD [12.11.4.1]**, shear in culverts shall be investigated in conformance with **LRFD [5.14.5.3]**. The location of the critical section for shear for culverts with haunches shall be determined in conformance with **LRFD [C5.13.3.6.1]** and shall be taken at a distance d_v from the end of the haunch.

36.3.4.3.1 Depth of Fill greater than or equal to 2.0 ft.

The shear resistance of the concrete, V_c, for <u>slabs</u> of box culverts with 2.0 feet or more of fill, for one-way action per **LRFD [5.14.5.3]** shall be determined as:

$$V_c = \left(0.0676\sqrt{f'_c} + 4.6\frac{A_s}{bd_e}\frac{V_u d_e}{M_u}\right)bd_e \le 0.126\sqrt{f'_c}\,bd_e$$

Where:

$$\frac{V_u d_e}{M_u} \le 1$$

Where:

V_c = Shear resistance of the concrete (kip)

A_s = Area of reinforcing steel in the design width (in^2)

d_e = Effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (in.)

V_u = Factored shear (kip)

M_u = Factored moment, occurring simultaneously with V_u (kip-in)

b = Design width (in.)

In the absence of shear reinforcing, the nominal shear resistance is equal to the shear resistance of the concrete. The factored resistance, V_r, or shear capacity, per **LRFD [5.8.2.1]** shall be taken as:

$$V_r = \phi V_n = \phi V_c$$

Per **LRFD [5.14.5.3]**, for single-cell box culverts only, Vc for slabs monolithic with walls need not be taken less than:

$$0.0948\sqrt{f'_c}bd_e$$

and V_c for slabs simply supported need not be taken less than:

$$0.0791\sqrt{f'_c}bd_e$$

The shear resistance of the concrete, V_c, for walls of box culverts with 2.0 feet or more of fill, for one-way action per **LRFD [5.8.3.3]** shall be determined as:

$$V_c = 0.0316\beta\sqrt{f'_c}b_v d_v \leq 0.25f'_c b_v d_v$$

Where:

V_c = Shear resistance of the concrete (kip)

β = 2.0 (**LRFD [5.8.3.4.1]**)

b_v = Effective web width taken as the minimum web width within the depth d_v (in.)

d_v = Effective shear depth as determined in **LRFD [5.8.2.9]**. Perpendicular distance between tension and compression resultants. Need not be taken less than the greater of $0.9d_e$ or $0.72h$ (in.)

36.3.4.3.2 Depth of Fill less than 2.0 ft

Per **LRFD [5.14.5.3]**, for box culverts with less than 2.0 feet of fill follow **LRFD [5.8]** and **LRFD [5.13.3.6]**.

The shear resistance of the concrete, V_c, for slabs and walls of box culverts with less than 2.0 feet of fill, for one-way action per **LRFD [5.8.3.3]** shall be determined as:

$$V_c = 0.0316\beta\sqrt{f'_c}\,b_v d_v \leq 0.25 f'_c\, b_v d_v$$

With variables defined above in 36.3.4.3.1.

For box culverts where the top slab is an integral part of the wearing surface (depth of fill equal zero) the top slab shall be checked for two-way action, as discussed in 18.3.3.2.2.

36.3.5 Service Limit State

Service I Limit State shall be applied as restrictions on stress, deformation, and crack width under regular service conditions **LRFD [1.3.2.2]**.

36.3.5.1 Factored Resistance

The resistance factor, ϕ, for Service Limit State, is found in **LRFD [1.3.2.1]** and its value is 1.00.

36.3.5.2 Crack Control Criteria

Per **LRFD [12.11.3]**, the provisions of **LRFD [5.7.3.4]** shall apply to crack width control in box culverts. All reinforced concrete members are subject to cracking under any load condition, which produces tension in the gross section in excess of the cracking strength of the concrete. Provisions are provided for the distribution of tension reinforcement to control flexural cracking.

Crack control criteria does not use a factored resistance, but calculates a maximum spacing for flexure reinforcement based on service load stress in bars, concrete cover and exposure condition.

Crack control criteria shall be applied when the tension in the cross-section exceeds 80% of the modulus of rupture, f_r, specified in **LRFD [5.4.2.6]** for Service I Limit State. The spacing, s, (in inches) of mild steel reinforcement in the layer closest to the tension face shall satisfy:

$$s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \quad \text{(in.)}$$

in which:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

Where:

γ_e	=	Exposure factor (1.0 for Class 1 exposure condition, 0.75 for Class 2 exposure condition, see **LRFD [5.7.3.4]** for guidance)
d_c	=	Thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.)
f_{ss}	=	Tensile stress in steel reinforcement at the service limit state (ksi)
h	=	Overall thickness or depth of the component (in.)

WisDOT Policy Item:

A class 1 exposure factor, γ_e = 1.0, shall be used for all cases for cast-in-place box culverts except for the top steel in the top slab of a box culvert with zero fill, where a class 2 exposure factor, γ_e = 0.75, shall be used.

36.3.6 Minimum Reinforcement Check

Per **LRFD [12.11.4.3]**, the area of reinforcement, A_s, in the box culvert cross-section should be checked for minimum reinforcement requirements per **LRFD [5.7.3.3.2]**.

The area of tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r, or moment capacity at least equal to the lesser of:

$1.2M_{cr}$ (or) $1.33M_u$

Where:

$$M_{cr} = \frac{f_r I_g}{c}$$

Where:

f_r	=	$0.37\sqrt{f'_c}$ Modulus of rupture (ksi) **LRFD [5.4.2.6]**
I_g	=	Gross moment of inertia (in^4)
c	=	½ *effective slab thickness (in.)

M_u = Total factored moment using Strength I Limit State (kip-in)

M_{cr} = Cracking strength moment (kip-in)

The factored resistance, M_r or moment capacity, shall be calculated as in 36.3.4.2 and shall satisfy:

$$M_r \geq \min(1.2M_{cr}, 1.33M_u)$$

36.3.7 Minimum Spacing of Reinforcement

Per **LRFD [5.10.3.1]**, the clear distance between parallel bars in a layer shall not be less than:

- 1.5 times the nominal diameter of the bars
- 1.5 times the maximum size of the course aggregate
- 1.5 inches

36.3.8 Maximum Spacing of Reinforcement

Per **LRFD [5.10.3.2]**, the spacing of reinforcement in walls and slabs shall not exceed:

- 1.5 times the thickness of the member
- 18 inches

36.3.9 Edge Beams

Per **LRFD [12.11.2.1]**, for cast-in-place box culverts, and for precast box culverts with top slabs having span to thickness ratios (s/t) > 18 or segment lengths < 4.0 feet, edge beams shall be provided as specified in **LRFD [4.6.2.1.4]** as follows:

- At ends of culvert runs where wheel loads travel within 24.0 inches from the end of the culvert
- At expansion joints of cast-in-place culverts where wheel loads travel over or adjacent to the expansion joint

The edge beam provisions are only applicable for culverts with less than 2.0 ft of fill, **LRFD [C12.11.2.1]**.

36.4 Design Loads

36.4.1 Self Weight (DC)

Include the structure self weight based on a unit weight of concrete of 0.150 kcf. When there is no fill on the top slab of the culvert, the top slab thickness includes a ½" wearing surface. The weight of the wearing surface is included in the design, but its thickness is not included in the section properties of the top slab. When designing the bottom slab of a culvert do not forget that the weight of the concrete in the bottom slab acts in an opposite direction than the bottom soil pressure and thus reduces the design moments and shears. This load is designated as, DC, dead load of structural components and nonstructural attachments, for application of load factors and limit state combinations.

36.4.2 Future Wearing Surface (DW)

If the fill depth over the culvert is greater than zero, the weight of the future wearing surface shall be taken as zero. If there is no fill depth over the culvert, the weight of the future wearing surface shall be taken as 20 psf. This load is designated as, DW, dead load of wearing surfaces and utilities, for application of load factors and limit state combinations.

36.4.3 Vertical and Horizontal Earth Pressure (EH and EV)

WisDOT Policy Item:

Box Culverts are assumed to be rigid frames. Use Vertical Earth Pressure load factors for rigid frames, in accordance with **LRFD [Table 3.4.1-2]**.

Use Horizontal Earth Pressure load factors for active soil pressure, in accordance with **LRFD [Table 3.4.1-2]**. Using load factors for active soil pressure is a conservative assumption.

The weight of soil above the buried structure is taken as 0.120 kcf. A coefficient of lateral earth pressure of 0.5 is used for the lateral pressure from the soil. This coefficient of lateral earth pressure is based on an at-rest condition and an effective friction angle of 30º, **LRFD [3.11.5.2]**. The lateral earth pressure is calculated per **LRFD [3.11.5.1]**:

$$p = k_o \gamma_s z$$

Where:

p	=	Lateral earth pressure (ksf)
k_o	=	Coefficient of at-rest lateral earth pressure
γ_s	=	Unit weight of backfill (kcf)
z	=	Depth below the surface of earth (ft)

WisDOT Policy Item:

For modification of earth loads for soil-structure interaction, embankment installations are always assumed for box culvert design, in accordance with **LRFD [12.11.2.2]**.

Soil-structure interaction for vertical earth loads is computed based on **LRFD [12.11.2.2]**. For embankment installations, the total unfactored earth load is:

$$W_E = F_e \gamma_s B_c H$$

In which:

$$F_e = 1 + 0.20\frac{H}{B_c}$$

Where:

W_E	=	Total unfactored earth load (kip/ft width)
F_e	=	Soil-structure interaction factor for embankment installations (F_e shall not exceed 1.15 for installations with compacted fill along the sides of the box section)
γ_s	=	Unit weight of backfill (kcf)
B_c	=	Outside width of culvert, as specified in Figure 36.4-1 (ft)
H	=	Depth of fill from top of culvert to top of pavement (ft)

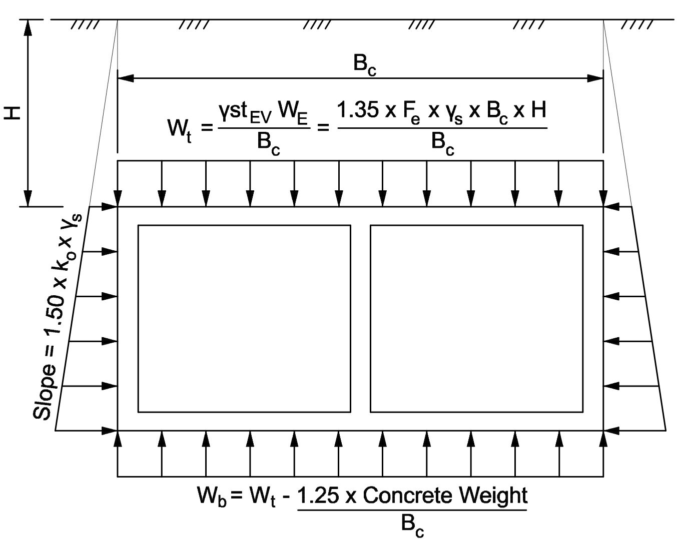


Figure 36.4-1
Factored Vertical and Horizontal Earth Pressures

Where:

W_t = Soil pressure on top of box culvert (ksf)

W_b = Soil pressure on the bottom of box culvert (ksf)

k_o = Coefficient of at-rest lateral earth pressure

γ_s = Unit weight of backfill (kcf)

Figure 36.4-1 shows the factored vertical and horizontal earth load pressures acting on a box culvert. The earth pressure from the dead load of the concrete is distributed equally over the bottom of the box.

36.4.4 Live Load Surcharge (LS)

Per **LRFD [3.11.6.4]**, a live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the distance from top of pavement to bottom of the box culvert.

Per **LRFD [Table 3.11.6.4-1]**, the following equivalent heights of soil for vehicular loading shall be used. The height to be used in the table shall be taken as the distance from the bottom of the culvert to the roadway surface. Use interpolation for heights other than those listed in the table.

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

Table 36.5-1
Equivalent Height of Soil for Vehicular Loading

Surcharge loads are computed based on a coefficient of lateral earth pressure times the unit weight of soil times the height of surcharge. A coefficient of lateral earth pressure of 0.5 is used for the lateral pressure from the soil, as discussed in 36.4.3. The uniform distributed load is applied to both exterior walls with the load directed toward the center of the box culvert. The load is designated as, LS, live load surcharge, for application of load factors and limit state combinations. Refer to **LRFD [3.11.6.4]** for additional information regarding live load surcharge.

36.4.5 Water Pressure (WA)

Static water pressure loads are computed when the water height on the outside of the box is greater than zero. The water height is measured from the bottom inside of the box culvert to the water level. The load is designated as, WA, water pressure load, for application of load factors and limit state combinations. Water pressure in culvert barrels is ignored. Refer to **LRFD [3.7.1]** for additional information regarding water pressure.

36.4.6 Live Loads (LL)

Live load consists of the standard AASHTO LRFD trucks and tandem. Per **LRFD [3.6.1.3.3]**, design loads are always axle loads (single wheel loads should not be considered) and the lane load is not used.

When the depth of fill over the box is less than 2 feet the wheel loads are distributed per **LRFD [4.6.2.10]**. When the depth of fill is 2 feet or more, the wheel loads are distributed per **LRFD [3.6.1.2.6]**. When areas from several concentrations overlap, the total load is considered as uniformly distributed over the area defined by the outside limits of the individual areas.

Per **LRFD [3.6.1.2.6]**, for single-span culverts, the effect of live load may be neglected when the depth of fill is more than 8.0 feet and exceeds the span length. For multiple span culverts, the effects may be neglected where the depth of fill exceeds the distance between faces of end walls.

Skew is not considered for design loads.

36.4.6.1 Depth of Fill less than 2.0 ft.

Where the depth of fill is less than 2.0 ft, follow **LRFD [4.6.2.10]**.

36.4.6.1.1 Case 1 – Traffic Travels Parallel to Span

When the traffic travels primarily parallel to the span, follow **LRFD [4.6.2.10.2]**. Use a single lane and the single lane multiple presence factor of 1.2.

Distribution length perpendicular to the span:

$$E = (96 + 1.44(S))$$

Where:

E	=	Equivalent distribution width perpendicular to span (in.)
S	=	Clear span (ft)

The distribution of wheel loads perpendicular to the span for depths of fill less than 2.0 feet is illustrated in Figure 36.4-2.

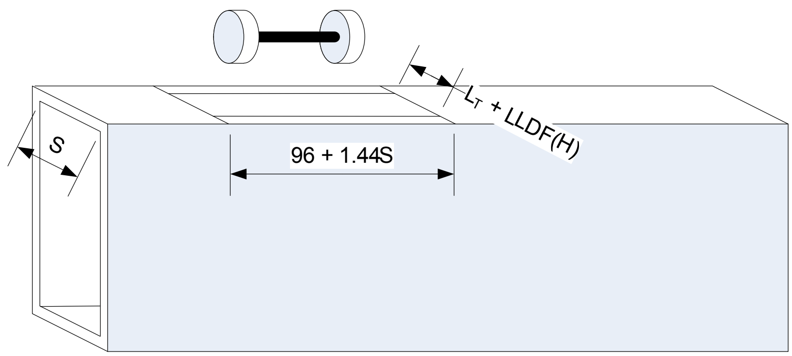


Figure 36.4-2
Distribution of Wheel Loads Perpendicular to Span, Depth of Fill Less than 2.0 feet

Distribution length parallel to the span:

$$E_{span} = (L_T + LLFD(H))$$

Where:

E_{span}	=	Equivalent distribution width parallel to span (in.)
L_T	=	Length of tire contact area parallel to span, as specified in **LRFD [3.6.1.2.5]** (in.)
LLDF	=	Factor for distribution of live load with depth of fill, 1.15, as specified in **LRFD [3.6.1.2.6]** for select granular backfill.
H	=	Depth of fill from top of culvert to top of pavement (in.)

The distribution of wheel loads parallel to the span for depths of fill less than 2.0 feet is illustrated in Figure 36.4-3.

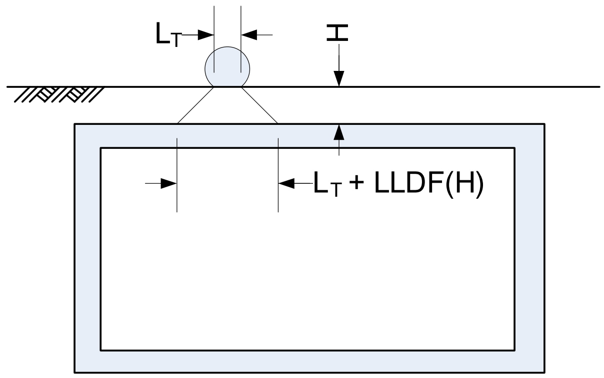


Figure 36.4-3
Distribution of Wheel Loads Parallel to Span, Depth of Fill Less than 2.0 feet

36.4.6.1.2 Case 2 - Traffic Travels Perpendicular to Span

When traffic travels perpendicular to the span, live load shall be distributed to the top slab using the equations specified in **LRFD [4.6.2.1]** for concrete decks with primary strips perpendicular to the direction of traffic. The effect of multiple lanes shall be considered. Use the multiple presence factor, m, as required per **LRFD [3.6.1.1.2]**.

For a cast-in-place box culvert, the width of the primary strip, in inches is:

+M: 26.0 + (6.6)(S)

-M: 48.0 + (3.0)(S)

Where:

S = Spacing of supporting components (ft)

+M = Positive moment

-M = Negative moment

36.4.6.2 Depth of Fill Greater than or Equal to 2.0 ft.

Where the depth of fill is 2.0 ft or greater, follow **LRFD [3.6.1.2.6]**. The effect of multiple lanes shall be considered. Use the multiple presence factor, m, as required per **LRFD [3.6.1.1.2]**.

WisDOT exception to AASHTO:

Where the depth of fill is 2.0 ft or greater, the live load distribution specified in **LRFD [3.6.1.2.6]** is not used in anticipation of a future ballot item. Use the following method for determining live load distribution.

The wheel loads are considered to be uniformly distributed over a rectangular area with sides equal to:

Longitudinal: $L_T + LLDF(H)$

Transverse: $W_T + LLDF(H) + 0.06(D)$

Where:

L_T	=	Length of tire contact area, per **LRFD [3.6.1.2.5]** (in.)
W_T	=	Width of tire contact area, per **LRFD [3.6.1.2.5]** (in.)
LLDF	=	1.15
H	=	Depth of fill from top of culvert to top of pavement (in.)
D	=	Interior span of the culvert (in.)

The longitudinal and transverse distribution widths for depths of fill greater than or equal to 2.0 feet are illustrated in Figure 36.4-4.

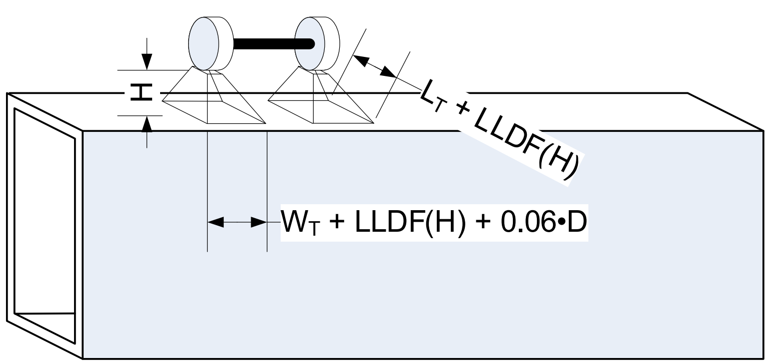


Figure 36.4-4
Distribution of Wheel Loads, Depth of Fill Greater than or Equal to 2.0 feet

36.4.7 Live Load Soil Pressures

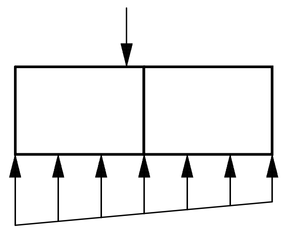


Figure 36.4-5
Vertical Soil Pressure under Culvert

The soil pressure on the bottom of the box is determined by moving the live load across the box. Find the location where the live load causes the maximum effects on the top slab of the box. At that location, determine the soil pressure diagram that will keep the system in equilibrium. Use the effects of this soil pressure in the bottom slab analysis.

36.4.8 Dynamic Load Allowance

Dynamic load allowance decreases as the depth of fill increases. **LRFD [3.6.2.2]** states that the impact on buried components shall be calculated as:

$$IM = 33(1.0 - 0.125(D_E)) \geq 0\%$$

Where:

D_E = Minimum depth of earth cover above the structure (ft)

36.4.9 Location for Maximum Moment

Create influence lines and use notional loading to determine the location for maximum moment. In this analysis, include cases for variable axle spacing and reverse axle order for unsymmetrical loading conditions.

For notional vehicles, only the portion of the loading that contributes to the effect being maximized is included. This is illustrated in Figure 36.4-6.

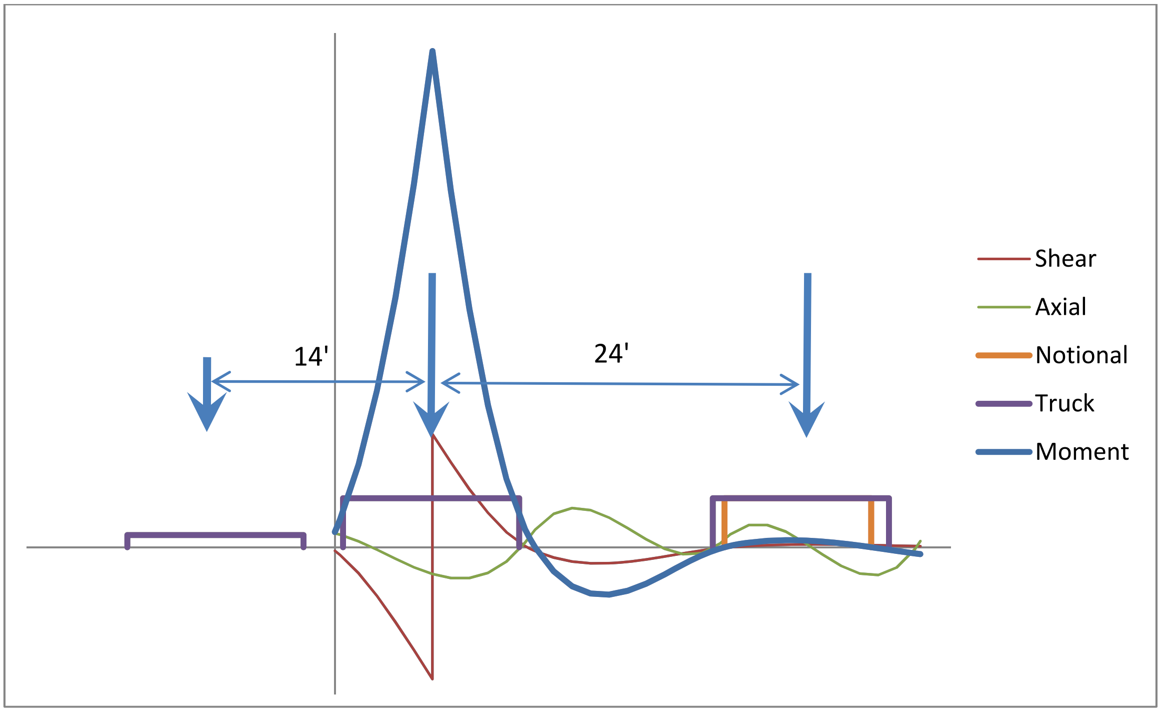


Figure 36.4-6
Application of Notional Loading using Influence Lines

The maximum positive moment results when the middle axial load is centered at the first positive peak while the variable rear axial spacing is 24 feet. Only the portion of the rear axial load in the positive region of the moment influence line is considered. The middle axial load and the portion of the rear axial in the positive region of the moment influence line are loaded on the shear and axial influence lines to compute the corresponding effects. Both positive and negative portions of the shear and axial influence lines are used when computing the corresponding effects. This process is repeated for maximizing the negative moment, shear and axial effects and computing the corresponding effects.

36.5 Design Information

Sidesway of the box is not considered because of the lateral support of the soil.

The centerline of the walls and top and bottom slabs are used for computing section properties and dimensions for analysis.

WisDOT Policy Item:

For skews 20 degrees or less, place the reinforcing steel along the skew. For skews over 20 degrees, place the reinforcing steel perpendicular to the centerline of box.

Culverts are analyzed as if the reinforcing steel is perpendicular to the centerline of box for all skew angles.

The minimum thickness of the top and bottom slab is 6½ inches. Minimum wall thickness is based on the inside opening of the box (height) and the height of the apron wall above the floor. Use the following table to select the minimum wall thickness that meets or exceeds the three criteria in the table.

Minimum Wall Thickness (Inches)	Cell Height (Feet)	Apron Wall Height Above Floor (Feet)
8	< 6	< 6.75
9	6 to < 10	6.75 to < 10
10	10 to > 10	10 to < 11.75
11		11.75 to < 12.5
12		12.5 to 13

Table 36.5-1
Minimum Wall Thickness Criteria

All slab thicknesses are rounded to the next largest ½ inch.

Top and bottom slab thicknesses are determined by shear and moment requirements. Slab thickness shall be adequate to carry the factored shear without shear reinforcement.

All bar steel is detailed as being 2 inches clear with the following exceptions:

- The bottom steel in the bottom slab is detailed with 3 inches clear
- The top steel in the top slab of a box culvert with no fill is detailed with 2½ inches clear

A haunch is provided only when the slab depth required at the interior wall is more than 2 inches greater than that required for the remainder of the span. Only 45° haunches shall be used. Minimum haunch depth and length is 6 inches. Haunch dimensions are increased in 3 inch increments.

The slab thickness required is determined by moment or shear, whichever governs.

The shear in the top and bottom slabs is assumed to occur at a distance "d" from the face of the walls. The value for "d" equals the distance from the centroid of the reinforcing steel to the face of the concrete in compression. When a haunch is used, shear must also be checked at the end of the haunch.

For multi-cell culverts make interior and exterior walls of equal thickness.

For culverts under high fills use a separate design for the ends if the reduced section may be used for at least two panel pours per end of culvert. Maximum length of panel pour is 40 feet.

Barrel lengths are based on the roadway sections and wing lengths are based on a minimum 2 1/2:1 slope of fill from the top of box to apron. Consideration shall be given to match the typical roadway cross slope.

Dimensions on drawings are given to the nearest ½ inch only.

36.6 Detailing of Reinforcing Steel

To calculate the required bar steel area and cutoff points a maximum positive and negative moment envelope is computed. It is assumed that the required bar lengths in the top slab are longer than those in the bottom slab. Therefore, cutoff points are computed for the top slab and are also used in the bottom slab.

36.6.1 Bar Cutoffs

Per **LRFD [5.11.1.2.1]**, all flexural reinforcement shall be extended beyond the point at which it is no longer required to resist flexure for a distance not less than:

- The effective depth of the member
- 15 times the nominal diameter of the bar
- 1/20 of the clear span

Continuing reinforcement shall extend not less than the development length, ℓ_d (**LRFD [5.11.2]**) beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure.

Per **LRFD [5.11.1.2.2]**, at least one-third of the positive moment reinforcement in simple span members and one-fourth of the positive moment reinforcement in continuous span members shall extend along the same face of the member beyond the centerline of the support. In beams, such extension shall not be less than 6.0 in.

Per **LRFD [5.11.1.2.3]**, at least one-third of the total tension reinforcement provided for negative moment at a support shall have an embedment length beyond the point of inflection not less than:

- The effective depth of the member
- 12 times the nominal diameter of the bar
- 0.0625 times the clear span

36.6.2 Corner Steel

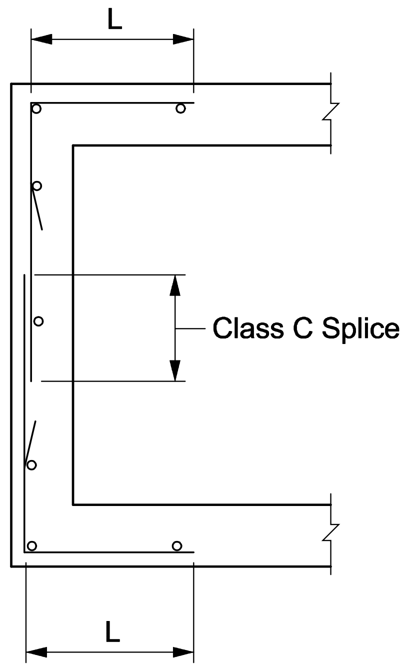


Figure 36.6-1
Layout of Corner Steel

The area of steel required is the maximum computed from using the top and bottom corner moments and the thickness of the slab or wall, whichever controls. Identical bars are used in the top and bottom corners. Identical length bars are used in the left and right corners if the bar lengths are within 2 feet of one another. Top and bottom negative steel is cut in the walls and detailed in two alternating lengths when a savings of over 2 feet in a single bar length can be obtained. Corner steel is always lapped at the center of the wall. If two bar lengths are used, only alternate bars are lapped.

Distance "L" is computed from the maximum negative moment envelope for the top slab and shall include the extension lengths discussed in 36.6.1.

36.6.3 Positive Moment Slab Steel

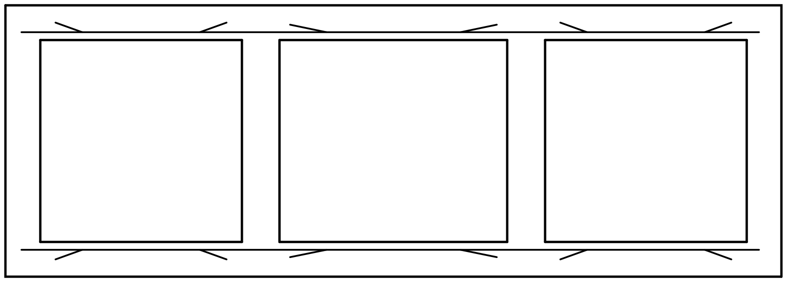


Figure 36.6-2
Layout of Positive Moment Steel

The area of steel required is determined by the maximum positive moments in each span. Top and bottom slab reinforcing steel may be of different size and spacing, but will have identical lengths. Detail two alternating bar lengths in a slab if 2 feet or more of bar steel can be saved in a single bar length.

When two alternating bar lengths are detailed in multi-cell culverts, run every other positive bar across the entire width of box. If this requires a length longer than 40 feet, lap them over an interior wall. For 2 or more cells, if the distance between positive bars of adjacent cells is 1 foot or less, make the bar continuous.

The cutoff points of alternate bars are determined from the maximum positive moment envelope for the top slab and shall include the extension lengths discussed in 36.6.1. These same points are used in the bottom slab. Identical bar lengths are used over multiple cells if bars are within 2 feet of one another.

36.6.4 Negative Moment Slab Steel over Interior Walls

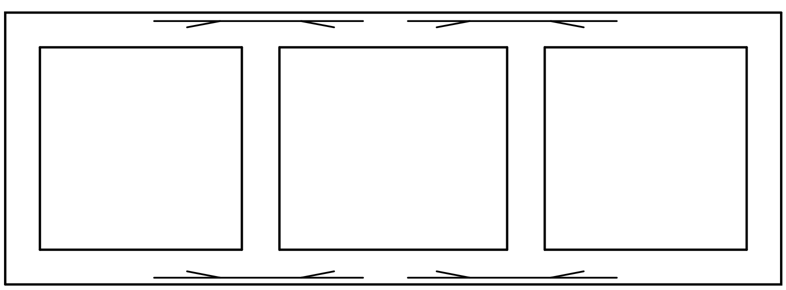


Figure 36.6-3
Layout of Negative Moment Steel

If no haunch is present, the area of steel required is determined by using the moment and effective depth at the face of the interior wall. If the slab is haunched, the negative reinforcement is determined per **LRFD [12.11.4.2],** which states that the negative moment is determined at the intersection of the haunch and uniform depth member. Top and bottom slab reinforcing steel may be of different size and spacing, but will have identical lengths. Detail two alternating bar lengths in a slab if 2 feet or more of bar steel can be saved in a single bar length.

Cutoff points are determined from the maximum negative moment envelope of the top slab and shall include the extension lengths discussed in 36.6.1. The same bar lengths are then used in the bottom slab. Identical bar lengths are used over multiple interior walls if bars are within 2 feet of one another. The minimum length of any bar is 2 times the development length. For culverts of 3 or more cells, if the clear distance between negative bars of adjacent spans is 1 foot or less, make the bar continuous across the interior spans.

When there is no fill over the top slab, run the negative moment reinforcing steel across the entire width of the culvert. Refer to 36.6.8 for temperature and shrinkage requirements.

36.6.5 Exterior Wall Positive Moment Steel

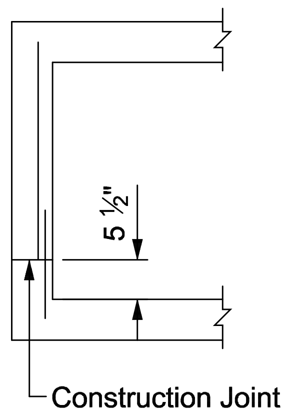


Figure 36.6-4
Layout of Exterior Wall Steel

The area of steel is determined by the maximum positive moment in the wall. A minimum of #4 bars at 18 inches is supplied. The wall bar is extended to 2 inch top clear and the dowel bar is extended to 3 inch bottom clear. A construction joint, 5 ½ inches above the bottom slab, is always used so a dowel bar must be detailed.

36.6.6 Interior Wall Moment Steel

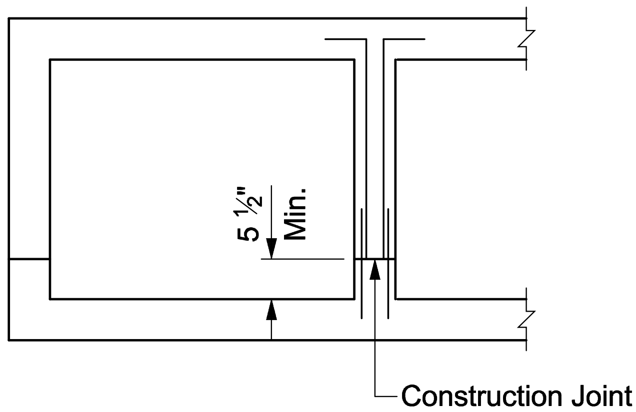


Figure 36.6-5
Layout of Interior Wall Steel

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

36.6.7 Distribution Reinforcement

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

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

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

Where:

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

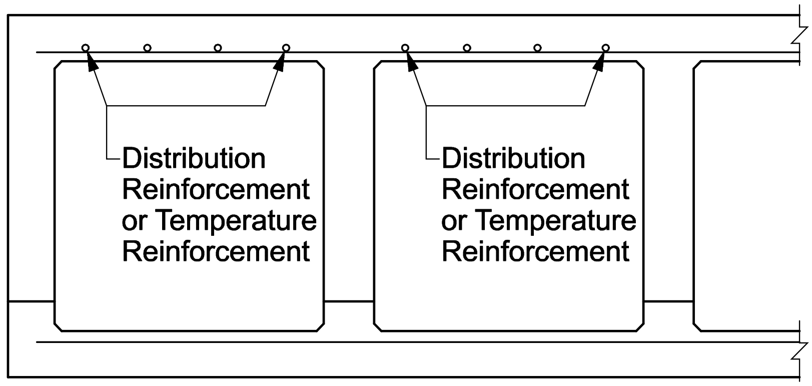


Figure 36.6-6
Layout of Distribution Steel

36.6.8 Temperature Reinforcement

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

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

$$0.11 \leq A_s \leq 0.60$$

Where:

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

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

Temperature steel is always #4 bars at a maximum spacing of 18 inches. When the top slab has not fill on top use a minimum of #4 bars at 12 inch centers in both directions in the top of the top slab.

36.7 Box Culvert Aprons

Five types of box culvert aprons are used. They are referred to as Type A, B, C, D and E. The angle that the wings make with the direction of stream flow is the main difference between the five types. The allowable headwater and other hydraulic requirements are what usually determine the type of apron required. Physical characteristics at the site may also dictate a certain type. For hydraulic design of different apron types see Chapter 8.

36.7.1 Type A

Type A, because of its poor hydraulic properties, is generally not used except for cattle or pedestrian underpasses.

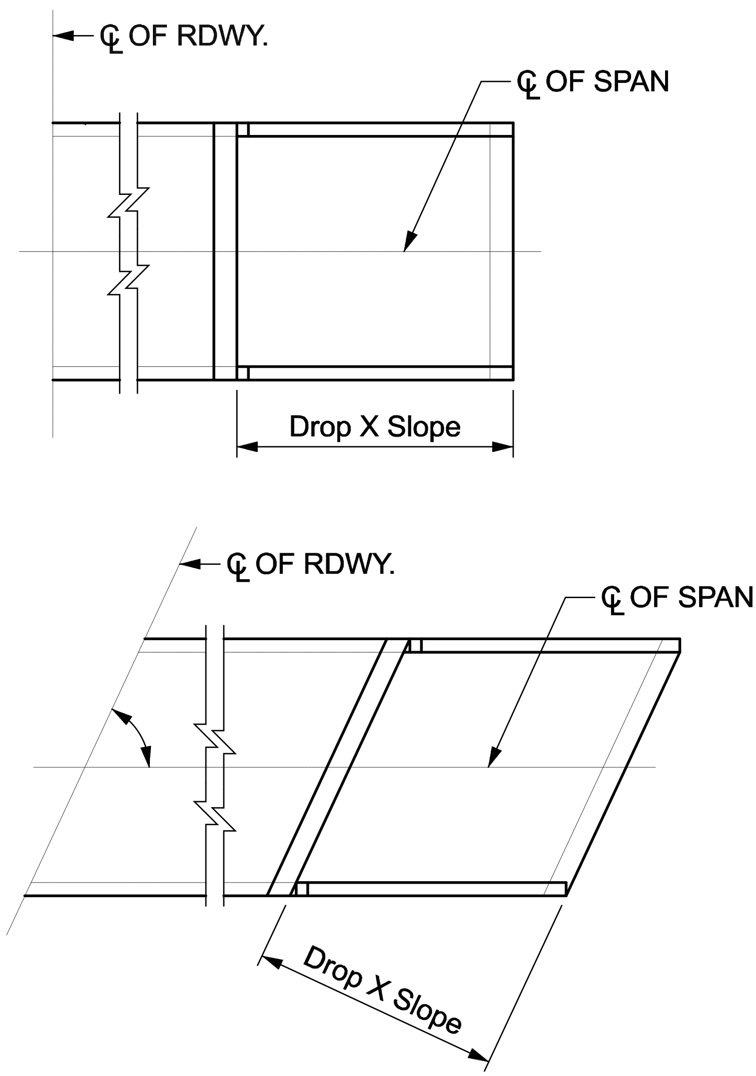
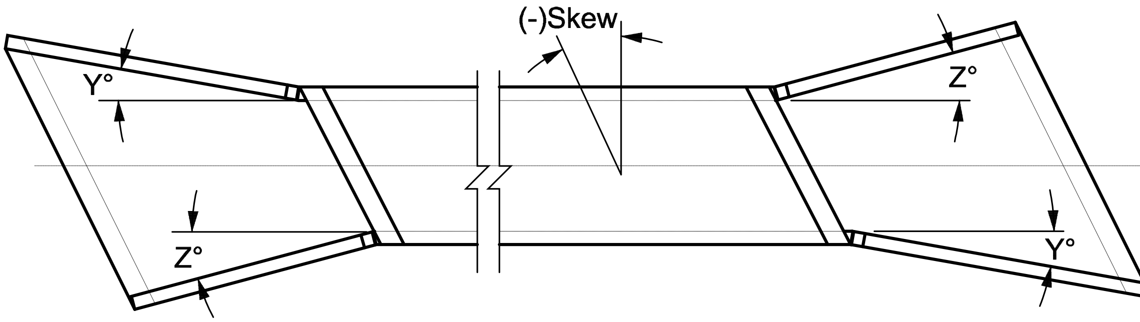


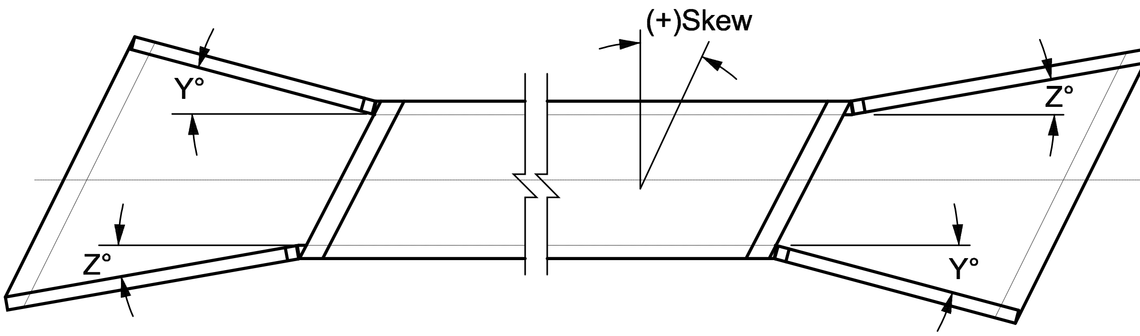
Figure 36.7-1
Plan View of Type A

36.7.2 Type B, C, D

Type B is used for outlets. Type C & D are of equal efficiency but Type C is used most frequently. Type D is used for inlets when the water is entering the culvert at a very abrupt angle. See Figure 36.7-2 for Wing Type B, C and D for guidance on wing angles for culvert skews.



Skew		Wing Type B		Wing Type C		Wing Type D	
Greater Than	To Include	Angle Y	Angle Z	Angle Y	Angle Z	Angle Y	Angle Z
0°	7.5°	15°	15°	30°	30°	45°	45°
7.5°	15.0°	15°	15°	25°	30°	40°	45°
15.0°	22.5°	10°	15°	25°	30°	35°	45°
22.5°	37.5°	10°	15°	20°	30°	30°	45°
37.5°	45.0°	10°	15°	15°	30°	25°	45°
45.0°	52.5°	5°	15°	15°	30°	20°	45°
52.5°	67.5°	5°	15°	10°	30°	15°	45°
67.5°	75.0°	5°	15°	5°	30°	10°	45°
75.0°	82.5°	0°	15°	5°	30°	5°	45°
82.5°	90.0°	0°	15°	0°	30°	0°	45°



Skew		Wing Type B		Wing Type C		Wing Type D	
Greater Than	To Include	Angle Y	Angle Z	Angle Y	Angle Z	Angle Y	Angle Z
0°	7.5°	15°	15°	30°	30°	45°	45°
7.5°	15.0°	15°	15°	30°	25°	45°	40°
15.0°	22.5°	15°	10°	30°	25°	45°	35°
22.5°	37.5°	15°	10°	30°	20°	45°	30°
37.5°	45.0°	15°	10°	30°	15°	45°	25°
45.0°	52.5°	15°	5°	30°	15°	45°	20°
52.5°	67.5°	15°	5°	30°	10°	45°	15°
67.5°	75.0°	15°	5°	30°	5°	45°	10°
75.0°	82.5°	15°	0°	30°	5°	45°	5°
82.5°	90.0°	15°	0°	30°	0°	45°	0°

Figure 36.7-2
Wing Type B, C, D (Angles vs. Skew)

36.7.3 Type E

Type E is used primarily in urban areas where a sidewalk runs over the culvert and it is necessary to have a parapet and railing along the sidewalk. For Type E the wingwalls run parallel to the roadway just like the abutment wingwalls of most bridges. It is also used where Right of Way (R/W) is a problem and the aprons would extend beyond the R/W for other types. Wingwall lengths for Type E wings are based on a minimum channel side slope of 1.5 to 1.

36.7.4 Wingwall Design

Culvert wingwalls are designed for a 1 foot surcharge, a unit weight of backfill of 0.120 kcf and a coefficient of lateral earth pressure of 0.5, as discussed in 36.4.3. Load and Resistance Factor Design is used, and the load factor for lateral earth pressure of γ_{EH} = 1.69 is used, based on past design experience. The lateral earth pressure was conservatively selected to keep wingwall deflection and cracking to acceptable levels. Many wingwalls that were designed for lower horizontal pressures have experienced excessive deflections and cracking at the footing. This may expose the bar steel to the water that flows through the culvert and if the water is of a corrosive nature, corrosion of the bar steel will occur. This phenomena has lead to complete failure of some wingwalls throughout the State.

Even with the increased steel the higher wings still deflected around ¾ inches at the top. To prevent this (in 1998) 1 inch diameter dowel bars are added between the wing and box wall for culverts over 6 feet high. The dowels have a bond breaker on the portion that extends into the wings.

For wing heights of 7 feet or less determine the area of steel required by using the maximum wall height and use the same bar size and spacing along the entire wingwall length. The minimum amount of steel used is #4 bars at 12 inch spacing. Wingwall thickness is made equal to the barrel wall thickness.

For wing heights over 7 feet the wall length is divided into two or more segments and the area of steel is determined by using the maximum height of each segment. Use the same bar size and spacing in each segment.

Wingwalls must satisfy Strength I Limit State for flexure and shear, and Service I Limit State for crack control, minimum reinforcement, and reinforcement spacing. Adequate shrinkage and temperature reinforcement shall be provided.

36.8 Box Culvert Camber

Camber of culverts is a design compensation for anticipated settlement of foundation soil beneath the culvert. Responsibility for the recommendation and calculation of camber belongs to the Regional Soils Engineer. Severe settlement problems with accompanying large camber are to be checked with the Geotechnical Section.

Both total and differential settlement need to be considered to determine the amount of box camber required to avoid adverse profile sag and undesirable separation at culvert joints per **LRFD[12.6.2.2]**. If the estimated settlement is excessive, contingency measures will need to be considered, such as preloading with embankment surcharge, undercutting and subgrade stabilization. To evaluate differential settlement, it will be necessary to calculate settlement at more than one point along the length of the box culvert.

36.8.1 Computation of Settlement

Settlement should be evaluated at the Service Limit state in accordance with **LRFD [12.6.2.2]** and **LRFD [10.6.2]**, and consider instantaneous elastic consolidation and secondary components. Elastic settlement is the instantaneous deformation of the soil mass that occurs as the soil is loaded. Consolidation settlement is the gradual compression of the soil skeleton when excess pore pressure is forced out of the voids in the soil. Secondary settlement, or creep, occurs as a result of plastic deformation of the soil skeleton under constant effective stress. Secondary settlement is typically not significant for box culvert design, except where there is an increase in effective stress within organic soil, such as peat. If secondary settlement is a concern, it should be estimated in accordance with **LRFD [10.6.2.4]**.

Total settlement, including elastic, consolidation and secondary components may be taken in accordance with **LRFD [10.6.2.4.1]** as:

$$S_t = S_e + S_c + S_s$$

Where:

S_t = Total settlement (ft)

S_e = Elastic settlement (ft)

S_c = Primary consolidation settlement (ft)

S_s = Secondary settlement (ft)

To compute settlement, the subsurface soil profile should be subdivided into layers based on stratigraphy to a depth of about 3 times the box width. The maximum layer thickness should be 10 feet.

Primary consolidation settlement for normally-consolidated soil is computed using the following equation in accordance with **LRFD [10.6.2.4.3]**:

$$S_c = \left[\frac{H_c}{1+e_o}\right] c_c \log_{10}\left[\frac{\sigma'_f}{\sigma'_o}\right]$$

Where:

S_c = Primary consolidation settlement (ft)

H_c = Initial height of compressible soil layer (ft)

e_o = Void ratio at initial vertical effective stress

C_c = Compression index which is a measure of the compressibility of a soil. It is the slope of the straight-line part of the e-log p curve from a conventional consolidation (oedometer) test.

σ'_f = Final vertical effective stress at midpoint of soil layer under consideration (ksf)

σ'_o = Initial vertical effective stress at midpoint of soil layer under consideration (ksf)

If the soil is overconsolidated, reference is made to **LRFD [10.6.2.4.3]** to estimate consolidation settlement.

Further description for the above equations and consolidation test can be found in most textbooks on soil mechanics.

For preliminary investigations C_c can be determined from the following approximate formula, found in most soil mechanics textbooks:

Non organic soils: $C_c = 0.007\ (LL-10)$

Where:

LL = Liquid limit expressed as whole number.

If the in-place moisture content approaches the plastic limit the computed C_c is decreased by 75%. If the in-place moisture content is near the liquid limit use the computed value. If the in-place moisture content is twice the liquid limit the computed C_c is increased by 75%. For intermediate moisture contents the percent change to the computed C_c is determined from a straight line interpolation between the corrections mentioned above.

If settlements computed by using the approximate value of C_c exceed 1.5 feet, a consolidation test is performed. As in-place moisture content approaches twice the liquid limit, settlement is caused by a local shear failure and the consolidation equation is no longer applicable.

The consolidation equation is applied to only compressible silts and clays. Sands are of a lower compressibility and no culvert camber is required until the fill exceeds 25 feet. When the fill exceeds 25 feet for sand, a camber of 0.01 feet per foot of fill is used.

36.8.2 Configuration of Camber

The following guides are to be followed when detailing camber.

- It is unnecessary to provide gradual camber. "Brokenback" camber is closer to the actual settlement which occurs.
- Settlement is almost constant from shoulder point to shoulder point. It then reduces to the ends of the culvert at the edge of the fill.
- The ends of the culvert tend to come up if side slopes are steeper than 2½ to 1. With 2 to 1 side slopes camber is increased 10% to compensate for this rise.

36.8.3 Numerical Example of Settlement Computation

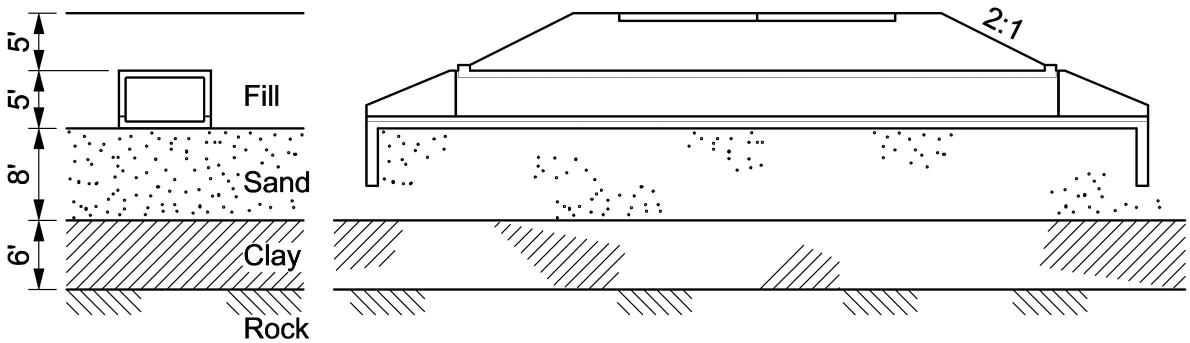


Figure 36.8-1
Soil Strata under Culvert

A box culvert rests on original ground consisting of 8 feet of sand and 6 feet of clay over bedrock. Estimate the settlement of the culvert if 10 feet of fill is placed on the original ground after the culvert is constructed. The in-place moisture content and liquid limit equal 40%. The initial void ratio equals 0.98. The unit weight of the clay is 105 pcf and that of the fill and sand is 110 pcf. There is no water table.

$$\sigma'_o = (8 \text{ ft})(110 \text{ pcf}) + (3 \text{ ft})(105 \text{ pcf}) = 1195 \text{ psf}$$

$$\sigma'_f = \sigma'_o + (10 \text{ ft})(110 \text{ pcf}) = 1195 \text{ psf} + 1100 \text{ psf} = 2295 \text{ psf}$$

$$C_c = 0.007\,(40\text{-}10) = 0.21 \text{ (approximate value)}$$

$$S_c = \left[\frac{H_c}{1+e_o}\right] c_c \log_{10}\left[\frac{\sigma'_f}{\sigma'_o}\right] = \frac{6\text{ft}}{1+0.98} 0.21 * \log_{10}\left[\frac{2295\text{psf}}{1195\text{psf}}\right] = 0.18\text{ft}$$

36.9 Box Culvert Structural Excavation and Structure Backfill

All excavations for culverts and aprons, unless on bedrock or fill, are undercut a depth of 6 inches. The upper limit of excavation is the existing ground line.

All spaces excavated and not occupied by the new structure are backfilled with structure backfill to the elevation and section existing prior to excavation within the length of the box. The backfill is placed to help eliminate settling problems on culverts. Backfill is placed in the undercut area under the apron. Usually 6 inches of structural backfill is placed under all boxes for construction purposes, which is covered by specification.

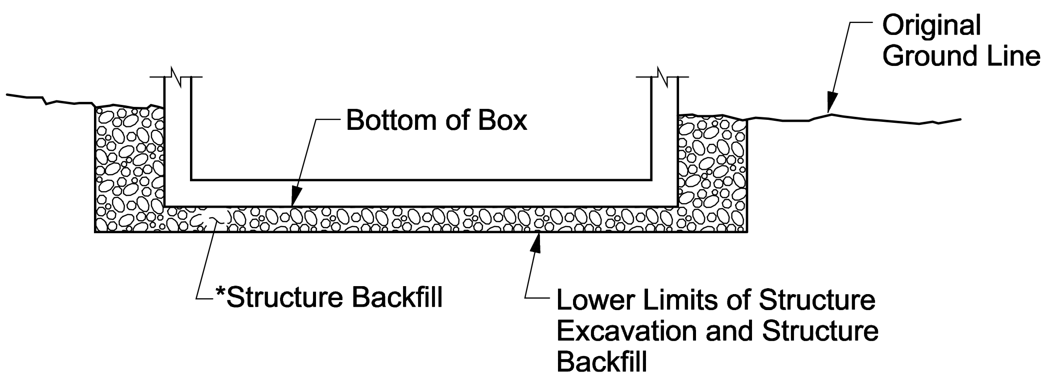


Figure 36.9-1
Limits for Excavation and Backfill

* Structure Backfill, No. 2 Washed Stone or Breaker Run Stone may be used to support culverts.

No backfill is placed under the box for culverts built on fills. The purpose of the backfill is to provide a solid base to pour the bottom slab. It is assumed that fill material provides this base without the addition of backfill.

36.10 Box Culvert Headers

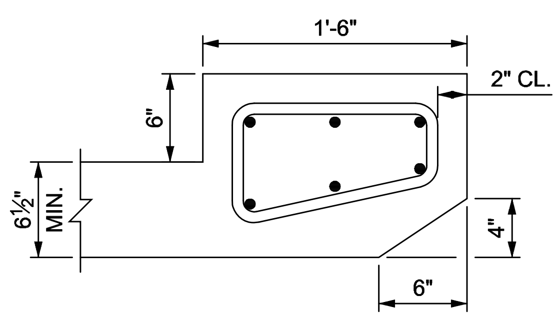
For skews of 20 degrees and less the main reinforcing steel is parallel to the end of the barrel. A header is not required for structural purposes but is used to prevent the fill material from spilling into the apron. A 12 inch wide by 6 inch high (above the top of top slab) header with nominal steel is therefore used for skews of 20 degrees and less on the top slab. No header is used on the bottom slab.

For skews over 20 degrees the main reinforcing is not parallel to the end of the barrel. The positive reinforcing steel terminates in the header and thus the header must support, in addition to its own dead load, an additional load from the dead load of the slab and fill above it. A portion of the live load may also have to be supported by the header.

The calculation of the actual load that a header must support becomes a highly indeterminate problem. For this reason a rational approach is used to determine the amount of reinforcement required in the headers. The design moment capacity of the header must be equal to or greater than 1.25 times the header dead load moment (based on simple span) plus 1.75 times a live load moment from a 16 kip load assuming 0.5 fixity at ends.

To prevent a traffic hazard, culvert headers are designed not to protrude above the ground line. For this reason the height of the header above the top of the top slab is allowed to be only 6 inches. The width of the header is standardized at 18 inches.

The header in the following figure gives the design moment capacities listed using d = 8.5 inches.



Bar Size	Moment Capacity ft.-kip
#6	46.1
#7	60.7
#8	76.5
#9	92.1
#10	109.1

Figure 36.10-1
Header Details (Skews > 20°)

The following size bars are recommended for the listed header lengths where "Header Length" equals the distance between C/L of walls in one cell measured along the skew.

Header Length	Bar Size [1]
To 11’	#7
Over 11’ to 14’	#8
Over 14’ to 17’	#9
Over 17’ to 20’	#10

Table 36.10-1
Header Reinforcement

[1] Use the bar size listed in each header and place 3 bars on the top and 3 bars on the bottom. Use a header on both the top and bottom slab. See the Standard *Box Culvert Details* in Chapter 36.

36.11 Plan Detailing Issues

36.11.1 Weep holes

Investigate the need for weep holes for culverts in cohesive soils. These holes are to relieve the hydrostatic pressure on the sides of the culverts. Where used, place the weep holes 1 foot above normal water elevation but a minimum of 1 foot above the lower sidewall construction joint. Do not place weep holes closer than 1 foot from the bottom of the top slab.

36.11.2 Cutoff Walls

Where dewatering the cutoff wall in sandy terrain is a problem, the concrete may be poured in the water. Place a note on the plans allowing concrete for the cutoff wall to be placed in the water.

36.11.3 Nameplate

Designate a location on the wingwall for placement of the nameplate. Locate nameplate on the first right wing traveling in the Cardinal direction (North/East).

36.11.4 Plans Policy

If a cast-in-place reinforced concrete box culvert is used, full plans must be provided and sealed by a professional engineer to the Bureau of Structures for approval. The plans must be in accordance with the *Bridge Manual* and Standards.

36.11.5 Rubberized Membrane Waterproofing

When required by the Standard Details, place the bid item "Rubberized Membrane Waterproofing" on the final plans. The quantity is given square yards.

36.12 Precast Four-Sided Box Culverts

In general, structural contractors prefer cast-in-place culverts while grading contractors prefer precast culverts. Precast culverts have been more expensive than cast-in-place culverts in the past, but allow for reduced construction time. Box culverts that are 4 feet wide by 6 feet high or less are considered roadway culverts. All other culverts require a B or C number along with the appropriate plans. All culverts requiring a number should be processed through the Bureau of Structures.

When a precast culvert is selected as the best structure type for a particular project during the design study phase, preliminary plans and complete detailed final plans are required to be sent to the Bureau of Structures for approval. The design and fabrication must be in accordance with ASTM Specification C1577, *AASHTO LRFD Specifications*, and the Bridge Manual.

Sometimes a complete set of plans is created for a cast-in-place culvert and a precast culvert is stated to be an acceptable alternate. If the contractor selects the precast alternate, the contractor is to submit shop drawings, sealed by a professional engineer, to the Bureau of Structures for approval. The design and fabrication must be in accordance with ASTM Specification C1577, *AASHTO LRFD Specifications*, and the Bridge Manual.

36.13 Three-Sided Structures

Three-sided box culvert structures are divided into two categories: cast-in-place three sided structures and precast three-sided structures. These structures shall follow the criteria outlined below.

36.13.1 Cast-In-Place Three-Sided Structures

To be developed

36.13.2 Precast Three-Sided Structures

Three-sided precast concrete structures offer a cost effective, convenient solution for a variety of bridge needs. The selection of whether a structure over a waterway should be a culvert, a three-sided precast concrete structure or a bridge is heavily influenced by the hydraulic opening. As the hydraulic opening becomes larger, the selection process for structure type progresses from culvert to three-sided precast concrete structure to bridge. Cost, future maintenance, profile grade, staging, skew, soil conditions and alignment are also important variables which should be considered. Culverts generally have low future maintenance; however, culverts should not be considered for waterways with a history or potential of debris to avoid channel cleanout maintenance. In these cases a three-sided precast concrete structure may be more appropriate. Three-sided precast concrete structures have the advantage of larger single and multiple openings, ease of construction, and low future maintenance costs.

A precast-concrete box culvert may be recommended by the Hydraulics Team. The side slope at the end or outcrop of a box culvert should be protected with guardrail or be located beyond the clear zone.

The hydraulic recommendations will include the Q_{100} elevation, the assumed flowline elevation, the required span, and the required waterway opening for all structure selections. The designer will determine the rise of the structure for all structure sections.

A cost comparison is required to justify a three-sided precast concrete structure compared to other bridge/culvert alternatives.

To facilitate the initiation of this type of project, the BOS is available to assist the Owners and Consultants in working out problems which may arise during plan development.

Some of the advantages of precast three-sided structures are listed below:

- Speed of Installation: Speed of installation is more dependent on excavation than product handling and placement. Precast concrete products arrive at the jobsite ready to install. Raw materials such as reinforcing steel and concrete do not need to be ordered, and no time is required on site to set up forms, place concrete, and wait for the concrete to cure. Precast concrete can be easily installed on-demand and immediately backfilled.

- Environmentally Friendly: Precast concrete is ready to be installed right off the delivery truck, which means less storage space needed for scaffolding and rebar. There is less noise pollution from ready-mix trucks continually pulling up on site and less waste as a result of using precast (i.e. no leftover steel, no pieces of scaffolding and no waste concrete piles). The natural bottom on a three-sided structure is advantageous to meet fish passage and DNR requirements.

- Quality Control: Because precast concrete products are produced in a quality-controlled environment with proper curing conditions, these products exhibit higher quality and uniformity over cast-in-place structures.

- Reduced Weather Dependency: Weather does not delay production of precast concrete as it can with cast-in-place concrete. Additionally, weather conditions at the jobsite do not significantly affect the schedule because the "window" of time required for installation is small compared to other construction methods, such as cast-in-place concrete.

- Maintenance: Single span precast three-sided structures are less susceptible to clogging from debris and sediment than multiple barrel culvers with equivalent hydraulic openings.

36.13.2.1 Precast Three-Sided Structure Span Lengths

WisDOT BOS allows and provides standard details for the following precast three-sided structure span lengths:

14’-0, 20’-0, 24’-0, 28’-0, 36’-0, 42’-0

Dimensions, rises, and additional guidance for each span length are provided in the standard details.

36.13.2.2 Segment Configuration and Skew

It is not necessary for the designer to determine the exact number and length of segments. The final structure length and segment configuration will be determined by the fabricator and may deviate from that implied by the plans.

A zero degree skew is preferable but skews may be accommodated in a variety of ways. Skew should be rounded to the nearer most-practical 5 deg., although the nearer 1 deg. is permissible where necessary. The range of skew is dependent on the design span and the fabrication limitations. Some systems are capable of fabricating a skewed segment up to a maximum of 45 degrees. Other systems accommodate skew by fabricating a special trapezoidal segment. If adequate right-of-way is available, skewed projects may be built with all right angle segments provided the angle of the wingwalls are adjusted accordingly. The designer shall consider the layout of the traffic lanes on staged construction projects when determining whether a particular three-sided precast concrete structure system is suitable.

Square segments are more economical if the structure is skewed. Laying out the structure with square segments will result in the greatest right-of-way requirement and thus allow ample space for potential redesign by the contractor, if necessary, to another segment configuration.

For a structure with a skew less than or equal to 15 deg., structure segments may be laid out square or skewed. Skewed segments are preferred for short structures (approximately less than 80 feet in length). Square segments are preferred for longer structures. However, skewed segments have a greater structural span. A structure with a skew of greater than 15 deg. requires additional analysis per the AASHTO LRFD Bridge Design Specifications. Skewed segments and the analysis both contribute to higher structure cost.

For a structure with a skew greater than 15 deg, structure segments should be laid out square. The preferred layout scheme for an arch-topped structure with a skew of greater than 15 deg should assume square segments with a sloping top of headwall to yield the shortest possible wingwalls. Where an arch-topped structure is laid out with skewed ends (headwalls parallel to the roadway), the skew will be developed within the end segments by varying the lengths of the legs as measured along the centerline of the structure. The maximum attainable skew is controlled by the difference between the full-segment leg length as recommended by the arch-topped-structure fabricator and a minimum leg length of 2 feet.

36.13.2.3 Minimum Fill Height

Minimum fill over a precast three-sided structure shall provide sufficient fill depth to allow adequate embedment for any required beam guard plus 6”. Refer to Standard 36.10 for further information.

Barriers mounted directly to the precast units are not allowed, as this connection has not been crash tested.

36.13.2.4 Rise

The maximum rises of individual segments are shown on the standard details. This limit is based on the fabrication forms and transportation. The maximum rise of the segment may also be limited by the combination of the skew involved because this affects transportation on the truck. Certain rise and skew combinations may still be possible but special permits may be required for transportation. The overall rise of the three-sided structure should not be a limitation when satisfying the opening requirements of the structure because the footing is permitted to extend above the ground to meet the bottom of the three-sided segment.

36.13.2.5 Deflections

Per **LRFD [2.5.2.6.2]**, the deflection limits for precast reinforced concrete three-sided structures shall be considered mandatory.

36.13.3 Plans Policy

If a precast or cast-in-place three-sided culvert is used, full design calculations and plans must be provided and sealed by a professional engineer to the Bureau of Structures for approval. The plans must be in accordance with the *Bridge Manual* and Standards.

The designer should use the span and rise for the structure selection shown on the plans as a reference for the information required on the title sheet. The structure type to be shown on the Title, Layout and General Plan sheets should be Precast Reinforced Concrete Three-Sided Structure.

The assumed elevations of the top of the footing and the base of the structure leg should be shown. For preliminary structure layout purposes, a 2-foot footing thickness should be assumed with the base of the structure leg seated 2 inches below the top-of-footing elevation. With the bottom of the footing placed at the minimum standard depth of 4 feet below the flow line elevation, the base of the structure leg should therefore be shown as 2'-2" below the flow line. An exception to the 4-foot depth will occur where the anticipated footing thickness is known to exceed 2 feet, where the footing must extend to rock, or where poor soil conditions and scour concerns dictate that the footing should be deeper.

The structure length and skew angle, and the skew, length and height of wingwalls should be shown. For a skewed structure, the wingwall geometrics should be determined for each wing. The sideslope used to determine the wing length should be shown on the plans.

If the height of the structure legs exceeds 10 feet, pedestals should be shown in the structure elevation view.

The following plan requirements shall be followed:

1. Preliminary plans are required for all projects utilizing a three-sided precast concrete structure.

2. Preliminary and Final plans for three-sided precast concrete structures shall identify the size (span x rise), length, and skew angle of the bridge.

3. Final plans shall include all geometric dimensions and a detailed design for the three-sided precast structure, all cast-in-place foundation units and cast-in-place or precast wingwalls and headwalls.

4. Final plans shall include the pay item Three-Sided Precast Concrete Structure and applicable pay items for the remainder of the substructure elements.

5. Final plans shall be submitted along with all pertinent special provisions to the BOS for review and approval.

In addition to foundation type, the wingwall type shall be provided on the preliminary and final plans. Similar to precast boxes, a wingwall design shall be provided which is supported independently from the three-sided structure. The restrictions on the use of cast-in-place or precast wings and headwalls shall be based on site conditions and the preferences of the Owner. These restrictions shall be noted on the preliminary and final plans.

36.13.4 Foundation Requirements

Precast and cast-in-place three-sided structures that are utilized in pedestrian or cattle underpasses can be supported on continuous spread or pile supported footings. Precast and cast-in-place three-sided structures that are utilized in waterway applications shall be supported on piling to prevent scour.

The footing should be kept level if possible. If the stream grade prohibits a level footing, the wingwall footings should be laid out to be constructed on the same plane as the structure footings. Continuity shall be established between the structural unit footing and the wingwall footing.

The allowable soil bearing pressure should be shown on the plans. Weak soil conditions could require pile foundations. If the footing is on piling, the nominal driving resistance should be shown. Where a pile footing is required, the type and size of pile and the required pile spacing, and which piles are to be battered, should be shown on the plans.

The geotechnical engineer should provide planning and design recommendations to determine the most cost effective and feasible foundation treatment to be used on the preliminary plans.

36.13.5 Precast Versus Cast-in-Place Wingwalls and Headwalls

The specifications for three-sided precast concrete structures permits the contractor to substitute cast-in-place for precast wingwalls and headwalls, and visa versa when cast-in-place is specified unless prohibited on the plans. Three-sided structures should be provided with adequate foundation support to satisfy the design assumptions permitting their relatively thin concrete section. These foundations are designed and provided in the plans. Spread footing foundations are most commonly used since they prove cost effective when rock or scour resistant soils are present with adequate bearing and sliding resistance. The use of precast spread footings shall be controlled by the planner and shall only be allowed when soil conditions permit and shall not be allowed to bear directly on rock or when rock is within 2 feet of the bottom of the proposed footing. When lower strength soils are present, or scour depths become large, a pile supported footing shall be used. The lateral loading design of the foundation is important because deflection of the pile or footing should not exceed the manufacturers' recommendations to preclude cracks developing.

36.14 Design Examples

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E36-1 Twin Cell Box Culvert LRFD

This example shows design calculations for a twin cell box culvert. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. *(Example is current through LRFD Fifth Edition - 2010 Interim)*

E36-1.1 Design Criteria

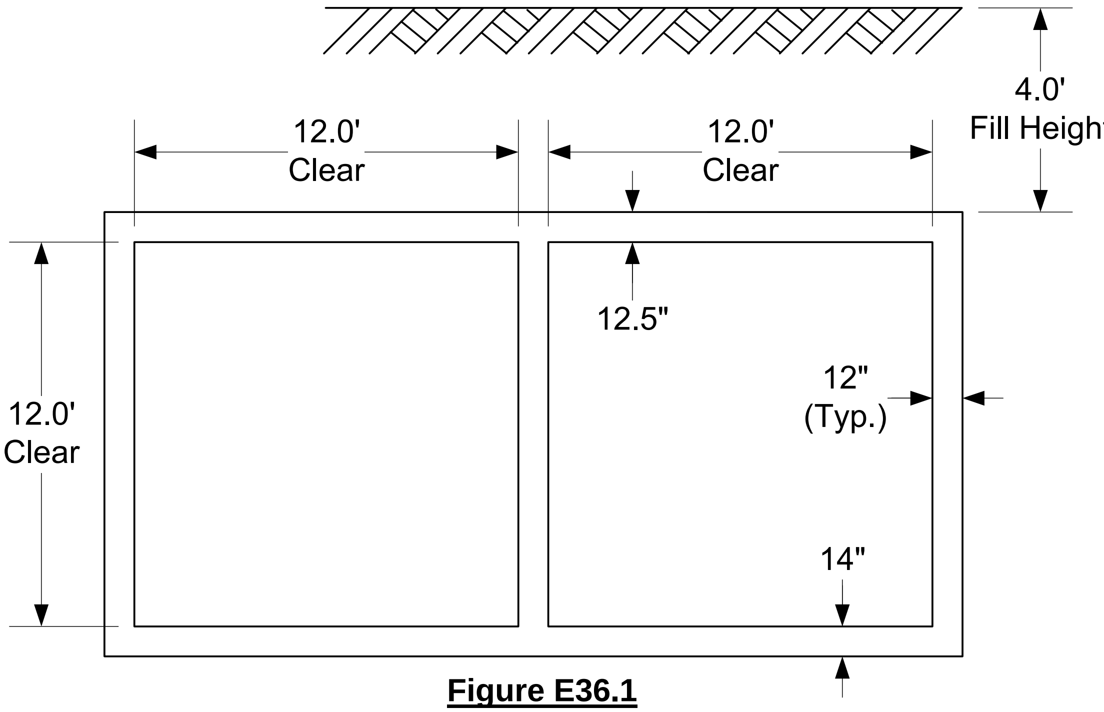


Figure E36.1
Box Culvert Dimensions

$NC = 2$ number of cells

$Ht = 12.0$ cell clear height, ft

$W_1 = 12.0$ cell 1 clear width, ft

$W_2 = 12.0$ cell 2 clear width, ft

$L = 134.0$ culvert length, ft

$t_{ts} = 12.5$ top slab thickness, in

$t_{bs} = 14.0$ bottom slab thickness, in

$t_{win} = 12.0$ interior wall thickness, in

$t_{wex} = 12.0$ exterior wall thickness, in

$H_{apron} := Ht + \frac{t_{ts}}{12}$ apron wall height above floor, ft

$H_{apron} = 13.04$

$f'_c := 3.5$	culvert concrete strength, ksi
$f_y := 60$	reinforcement yield strength, ksi
$E_s := 29000$	modulus of elasticity of steel, ksi
$skew = 0.0$	skew angle, degrees
$H_s = 4.00$	depth of backfill above top edge of top slab, ft
$w_c := 0.150$	weight of concrete, kcf
$cover_{bot} := 3$	concrete cover (bottom of bottom slab), in
$cover := 2$	concrete cover (all other applications), in
$LS_{ht} := 2.2$	live load surcharge height, ft

Resistance factors, reinforced concrete cast-in-place box structures, **LRFD [Table 12.5.5-1]**

$\phi_f := 0.9$	resistance factor for flexure
$\phi_v := 0.85$	resistance factor for shear

Calculate the span lengths for each cell (measured between centerlines of walls)

$$S_1 := W_1 + \frac{1}{12}\left(\frac{t_{win}}{2} + \frac{t_{wex}}{2}\right) \qquad \boxed{S_1 = 13.00} \text{ ft}$$

$$S_2 = W_2 + \frac{1}{12}\cdot\left(\frac{t_{wex}}{2} + \frac{t_{win}}{2}\right) \qquad \boxed{S_2 = 13.00} \text{ ft}$$

Verify that the box culvert dimensions fall within WisDOT's minimum dimension criteria. Per **[36.2]**, the minimum size for pedestrian underpasses is 8 feet high by 5 feet wide. The minimum size for cattle underpass is 6 feet high by 5 feet wide. A minimum height of 5 feet is desirable for cleanout purposes.

Does the culvert meet the minimum dimension criteria? $\boxed{check = \text{"OK"}}$

Verify that the slab and wall thicknesses fall within WisDOT's minimum dimension criteria. Per **[36.5]**, the minimum thickness of the top and bottom slab is 6.5 inches. Per **[Table 36.5-1]**, the minimum wall thickness varies with respect to cell height and apron wall height.

Minimum Wall Thickness (Inches)	Cell Height (Feet)	Apron Wall Height Above Floor (Feet)
8	< 6	< 6.75
9	6 to < 10	6.75 to < 10
10	10 to > 10	10 to < 11.75
11		11.75 to < 12.5
12		12.5 to 13

Table 36.5-1
Minimum Wall Thickness Criteria

Do the slab and wall thicknesses meet the minimum dimension criteria? $\boxed{check = \text{"OK"}}$

Since this example has more than 2.0 feet of fill, edge beams are not required.

E36-1.2 Modulus of Elasticity of Concrete Material

Per **[9.2]**, use $f`_c$ = 3.5 ksi for culverts. The value of E is calculated per **LRFD [5.4.2.4]**:

$\boxed{K_1 := 1}$ $E_{c\_calc} := 33000 \cdot K_1 \cdot w_c^{1.5} \cdot \sqrt{f'_c} = 3586.6$ ksi

$E_c := 3600$ ksi modulus of elasticity of concrete, per **[9.2]**

E36-1.3 Loads

$\gamma_s := 0.120$ unit weight of soil, kcf

Per **[36.5]**, a haunch is provided only when the slab depth required at the interior wall is more than 2 inches greater than that required for the remainder of the span. Minimum haunch depth and length is 6 inches. Haunch depth is increased in 3 inch increments. For the first iteration, assume there are no haunches.

$h_{hau} := 0.0$ haunch height, in

$l_{hau} := 0.0$ haunch length, in

$wt_{hau} = 0.0$ weight of one haunch, kip

E36-1.3.1 Dead Loads

Dead load (DC):

top slab dead load:

$$w_{dlts} := w_c \cdot \frac{t_{ts}}{12} \cdot 1 \qquad \boxed{w_{dlts} = 0.156} \text{ klf}$$

bottom slab dead load:

$$w_{dlbs} := w_c \cdot \frac{t_{bs}}{12} \cdot 1 \qquad \boxed{w_{dlbs} = 0.175} \text{ klf}$$

Wearing Surface (DW):

Per **[36.4.2]**, the weight of the future wearing surface is zero if there is any fill depth over the culvert. If there is no fill depth over the culvert, the weight of the future wearing surface shall be taken as 0.020 ksf.

$w_{ws} = 0.000$ weight of future wearing surface, ksf

Vertical Earth Load (EV):

Calculate the modification of earth loads for soil-structure interaction per **LRFD [12.11.2.2]**. Per the policy item in **[36.4.3]**, embankment installations are always assumed.

Installation_Type = "Embankment"

$\gamma_s = 0.120$ unit weight of soil, kcf

$B_c = 27.00$ outside width of culvert, ft
(measured between outside faces of exterior walls)

$H_s = 4.00$ depth of backfill above top edge of top slab, ft

Calculate the soil-structure interaction factor for embankment installations:

$$F_e := 1 + 0.20 \cdot \frac{H_s}{B_c} \qquad \boxed{F_e = 1.03}$$

F_e shall not exceed 1.15 for installations with compacted fill along the sides of the box section:

$$\boxed{F_e = 1.03}$$

Calculate the total unfactored earth load:

$$W_E := F_e \cdot \gamma_s \cdot B_c \cdot H_s \qquad \boxed{W_E = 13.34} \text{ klf}$$

Distrubute the total unfactored earth load to be evenly distributed across the top of the culvert:

$$w_{sv} := \frac{W_E}{B_c} \qquad \boxed{w_{sv} = 0.494}$$

Horizontal Earth Load (EH):

soil horizontal earth load (magnitude at bottom and top of wall):

$k_o := 0.5$ coefficient of at rest lateral earth pressure **[36.4.3]**

$\gamma_s = 0.120$ unit weight of soil, kcf

$$w_{sh\_bot} := k_o \cdot \gamma_s \cdot \left(Ht + \frac{t_{ts}}{12} + \frac{t_{bs}}{12} + H_s\right) \cdot 1 \qquad \boxed{w_{sh\_bot} = 1.09} \text{ klf}$$

$$w_{sh\_top} := k_o \cdot \gamma_s \cdot (H_s) \cdot 1 \qquad \boxed{w_{sh\_top} = 0.24} \text{ klf}$$

Live Load Surcharge (LS):

soil live load surcharge:

$k_o = 0.5$ coefficient of lateral earth pressure

$\gamma_s = 0.120$ unit weight of soil, kcf

$LS_{ht} = 2.2$ live load surcharge height per **[36.4.4]**, ft

$$w_{sll} := k_o \cdot \gamma_s \cdot LS_{ht} \cdot 1 \qquad \boxed{w_{sll} = 0.13} \text{ klf}$$

E36-1.3.2 Live Loads

For Strength 1 and Service 1:

HL-93 loading = design truck (no lane) **LRFD [3.6.1.3.3]**
design tandem (no lane)

For the Wisconsin Standard Permit Vehicle (Wis-SPV) Check:

The Wis-SPV vehicle is to be checked during the design phase to make sure it can carry a minimum vehicle load of 190 kips. The current version of AASHTO LRFR does not cover rating of concrete box culverts.

E36-1.4 Live Load Distribution

Live loads are distributed over an equivalent area, with distribution components both parallel and perpendicular to the span, as calculated below. Per **LRFD [3.6.1.3.3]**, the live loads to be placed on these widths are axle loads (i.e., two lines of wheels) without the lane load. The equivalent distribution width applies for both live load moment and shear.

E36-1.5 Equivalent Strip Widths for Box Culverts

The calculations for depths of fill less than 2.0 ft, per **LRFD [4.6.2.10]** are not required for this example. The calcuations are shown for illustration purposes only.

The calculations below follow **LRFD [4.6.2.10.2]** - Case 1: Traffic Travels Parallel to Span. If traffic travels perpendicular to the span, follow **LRFD [4.6.2.10.3]** - Case 2: Traffic Travels Perpendicular to Span, which states to follow **LRFD [4.6.2.1]**.

Per **LRFD [4.6.2.10.2]**, when traffic travels primarily parallel to the span, culverts shall be analyzed for a single loaded lane with a single lane multiple presence factor.

Therefore, $mpf = 1.2$

Perpendicular to the span:

It is conservative to use the largest distribution factor from each span of the structure across the entire length of the culvert. Therefore, use the smallest span to calculate the smallest strip width. That strip width will provide the largest distribution factor.

$S := \min(W_1, W_2)$ clear span, ft $\boxed{S = 12.00}$ ft

The equivalent distribution width perpendicular to the span is:

$$E_{perp} := \frac{1}{12} \cdot (96 + 1.44 \cdot S) \qquad \boxed{E_{perp} = 9.44} \text{ ft}$$

Parallel to the span:

$H_s = 4.00$ depth of backfill above top edge of top slab, ft

$L_T := 10$ length of tire contact area, in **LRFD [3.6.1.2.5]**

$LLDF = 1.15$ live load distribution factor. From **LRFD [4.6.2.10.2]**, LLDF = 1.15 as specified in **LRFD [3.6.1.2.6]** for select granular backfill

The equivalent distribution width parallel to the span is:

$$E_{parallel} := \frac{1}{12} \cdot (L_T + LLDF \cdot H_s \cdot 12) \qquad \boxed{E_{parallel} = 5.43} \text{ ft}$$

The equivalent distribution widths parallel and perpendicular to the span create an area that the axial load shall be distributed over. The equivalent area is:

$$E_{area} := E_{perp} \cdot E_{parallel} \qquad \boxed{E_{area} = 51.29} \text{ ft}^2$$

For depths of fill 2.0 ft. or greater calculate the size of the rectangular area that the wheels are considered to be uniformly distributed over, per **[36.4.6.2] Exception to AASHTO**.

$L_T = 10.0$ length of tire contact area, in **LRFD [3.6.1.2.5]**

$W_T := 20$ width of tire contact area, in **LRFD [3.6.1.2.5]**

The length and width of the equivalent area for 1 wheel are:

$L_{eq\_i} := L_T + LLDF \cdot H_s \cdot 12$ $\boxed{L_{eq\_i} = 65.20}$ in

$W_{eq\_i} := W_T + LLDF \cdot H_s \cdot 12 + 0.06 \cdot \max(W_1, W_2) \, 12$ $\boxed{W_{eq\_i} = 83.84}$ in

Where such areas from several wheels overlap, the total load shall be uniformly distributed over the area, **LRFD [3.6.1.2.6]**.

Check if the areas overlap = "Yes, the areas overlap" therefore, use the following length and width values for the equivalent area for 1 wheel:

	Front and Rear Wheels:		Center Wheel:	
Length	$L_{eq13} = 65.2$	in	$L_{eq2} = 65.2$	in
Width	$W_{eq13} = 77.9$	in	$W_{eq2} = 77.9$	in
Area	$A_{eq13} = 5080.4$	in^2	$A_{eq2} = 5080.4$	in^2

Per **LRFD [3.6.1.2.2]**, the weights of the design truck wheels are below. (Note that one axle load is equal to two wheel loads.)

$W_{wheel1i} := 4000$ front wheel weight, lbs

$W_{wheel23i} := 16000$ center and rear wheel weights, lbs

The effect of single and multiple lanes shall be considered. For this problem, a single lane with the single lane multiple presence factor governs. Applying the single lane multiple presence factor:

$W_{wheel1} := mpf \cdot W_{wheel1i} = 4800$ lbs $mpf = 1.20$

$W_{wheel23} := mpf \cdot W_{wheel23i} = 19200$ lbs

For single-span culverts, the effects of the live load may be neglected where the depth of fill is more than 8.0 feet and exceeds the span length. For multiple span culverts, the effects of the live load may be neglected where the depth of fill exceeds the distance between faces of endwalls, **LRFD [3.6.1.2.6]**.

Note: The wheel pressure values shown here are for the 14'-0" variable axle spacing of the design truck, which controls over the design tandem for this example. In general, all variable axle spacings of the design truck and the design tandem must be investigated to account for the maximum response.

$LL1 = 0.94$ live load pressure (front wheel), psi

$LL2 = 3.78$ live load pressure (center wheel), psi

$LL3 = 3.78$ live load pressure (rear wheel), psi

E36-1.6 Limit States and Combinations

The limit states, load factors and load combinations shall be applied as required and detailed in chapter 36 of this manual and as indicated below.

E36-1.6.1 Load Factors

From **LRFD [Table 3.4.1-1]** and **LRFD [Table 3.4.1-2]**:

Per the policy item in **[36.4.3]** Assume box culverts are closed, rigid frames. Assume active earth pressure to be conservative.

	Strength 1	Service 1
DC	$\gamma st_{DCmax} := 1.25$	$\gamma s1_{DC} := 1.0$
	$\gamma st_{DCmin} := 0.9$	
DW	$\gamma st_{DWmax} := 1.5$	$\gamma s1_{DW} := 1.0$
	$\gamma st_{DWmin} := 0.65$	
EV	$\gamma st_{EVmax} := 1.35$	$\gamma s1_{EV} := 1.0$
	$\gamma st_{EVmin} := 0.9$	
EH	$\gamma st_{EHmax} := 1.50$	$\gamma s1_{EH} := 1.0$
	$\gamma st_{EHmin} := 0.5$ **LRFD [3.11.7]**	
LS	$\gamma st_{LSmax} := 1.75$	$\gamma s1_{ES} := 1.0$
	$\gamma st_{LSmin} := 0$	
LL	$\gamma st_{LL} := 1.75$	$\gamma s1_{LL} := 1.0$

Dynamic Load Allowance (IM) is applied to the truck and tandem. From **LRFD [3.6.2.2]**, IM of buried components varies with depth of cover above the structure and is calculated as:

$IM := 33 \cdot \left(1.0 - 0.125 \cdot H_S\right)$ (where H_S is in feet) $IM = 16.50$

If IM is less than 0, use IM = 0 $\boxed{IM = 16.50}$

E36-1.6.2 Dead Load Moments and Shears

The unfactored dead load moments and shears for each component are listed below (values are per 1-foot width and are in kip-ft and kip, respectively):

Exterior Wall Unfactored Dead Load Moments (kip-ft)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	-1.52	-1.44	-5.14	-1.01	0.00
0.1	-1.42	-1.54	-0.12	-0.14	0.00
0.2	-1.31	-1.63	3.53	0.55	0.00
0.3	-1.21	-1.73	5.92	1.04	0.00
0.4	-1.10	-1.82	7.14	1.34	0.00
0.5	-1.00	-1.91	7.30	1.46	0.00
0.6	-0.89	-2.01	6.51	1.38	0.00
0.7	-0.79	-2.10	4.87	1.12	0.00
0.8	-0.68	-2.19	2.49	0.66	0.00
0.9	-0.58	-2.29	-0.54	0.01	0.00
1.0	-0.48	-2.38	-4.11	-0.82	0.00

Interior Wall Unfactored Dead Load Moments (kip-ft)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	0.00	0.00	0.00	0.00	0.00
0.1	0.00	0.00	0.00	0.00	0.00
0.2	0.00	0.00	0.00	0.00	0.00
0.3	0.00	0.00	0.00	0.00	0.00
0.4	0.00	0.00	0.00	0.00	0.00
0.5	0.00	0.00	0.00	0.00	0.00
0.6	0.00	0.00	0.00	0.00	0.00
0.7	0.00	0.00	0.00	0.00	0.00
0.8	0.00	0.00	0.00	0.00	0.00
0.9	0.00	0.00	0.00	0.00	0.00
1.0	0.00	0.00	0.00	0.00	0.00

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

Top Slab Unfactored Dead Load Shears (kip)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	0.74	2.45	0.67	0.13	0.00
0.1	0.55	1.86	0.67	0.13	0.00
0.2	0.36	1.26	0.67	0.13	0.00
0.3	0.17	0.67	0.67	0.13	0.00
0.4	-0.01	0.08	0.67	0.13	0.00
0.5	-0.20	-0.52	0.67	0.13	0.00
0.6	-0.39	-1.11	0.67	0.13	0.00
0.7	-0.58	-1.70	0.67	0.13	0.00
0.8	-0.76	-2.30	0.67	0.13	0.00
0.9	-0.95	-2.89	0.67	0.13	0.00
1.0	-1.14	-3.48	0.67	0.13	0.00

Bottom Slab Unfactored Dead Load Shears (kip)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	1.86	2.32	0.94	0.16	0.00
0.1	1.40	1.73	0.94	0.16	0.00
0.2	0.94	1.14	0.94	0.16	0.00
0.3	0.48	0.54	0.94	0.16	0.00
0.4	0.02	-0.05	0.94	0.16	0.00
0.5	-0.44	-0.64	0.94	0.16	0.00
0.6	-0.90	-1.24	0.94	0.16	0.00
0.7	-1.36	-1.83	0.94	0.16	0.00
0.8	-1.82	-2.42	0.94	0.16	0.00
0.9	-2.28	-3.01	0.94	0.16	0.00
1.0	-2.74	-3.61	0.94	0.16	0.00

The DC values are the component dead loads and include the self weight of the culvert and haunch (if applicable).

The DW values are the dead loads from the future wearing surface (DW values occur only if there is no fill on the culvert).

The EV values are the vertical earth loads from the fill on top of the box culvert.

The EH values are the horizontal earth loads from the fill on the sides of the box culvert.

The LS values are the live load surcharge loads (assuming $LS_{ht} = 2.2$ feet of surcharge)

E36-1.6.3 Live Load Moments and Shears

The unfactored live load load moments and shears (per lane including impact) are listed below (values are in kip-ft and kips, respectively). A separate analysis run will be required if results without impact are desired.

Exterior Wall
Unfactored Live Load Moments (kip-ft)

Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.73	-1.74	0.74	-1.77
0.1	0.67	-1.70	0.69	-1.92
0.2	0.61	-1.67	0.65	-2.07
0.3	0.55	-1.65	0.62	-2.21
0.4	0.48	-1.68	0.60	-2.36
0.5	0.42	-1.82	0.58	-2.51
0.6	0.37	-1.97	0.56	-2.69
0.7	0.41	-2.12	0.56	-2.86
0.8	0.47	-2.28	0.61	-3.04
0.9	0.55	-2.44	0.68	-3.21
1.0	0.65	-2.61	0.77	-3.39

Interior Wall
Unfactored Live Load Moments (kip-ft)

Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.99	-0.99	0.88	-0.88
0.1	0.93	-0.93	0.99	-0.99
0.2	0.92	-0.92	1.12	-1.12
0.3	0.90	-0.90	1.25	-1.25
0.4	0.90	-0.90	1.38	-1.38
0.5	1.08	-1.08	1.54	-1.53
0.6	1.27	-1.27	1.74	-1.74
0.7	1.47	-1.47	1.99	-1.99
0.8	1.69	-1.69	2.24	-2.24
0.9	1.92	-1.92	2.50	-2.50
1.0	2.17	-2.17	2.75	-2.75

Top Slab Unfactored Live Load Moments (kip-ft)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.81	-1.76	0.65	-2.16
0.1	2.24	-0.34	1.83	-0.20
0.2	3.81	-0.27	4.23	-0.32
0.3	5.06	-0.49	5.92	-0.66
0.4	5.71	-0.75	6.78	-1.04
0.5	5.76	-1.04	6.90	-1.43
0.6	5.22	-1.34	6.21	-1.82
0.7	4.13	-1.64	4.74	-2.22
0.8	2.56	-1.96	2.54	-2.62
0.9	0.86	-3.59	0.76	-3.02
1.0	0.07	-5.89	0.06	-4.81

Bottom Slab Unfactored Live Load Moments (kip-ft)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.46	-0.67	0.40	-0.35
0.1	1.72	-0.29	2.52	-0.32
0.2	3.30	-0.76	4.46	-0.78
0.3	4.25	-1.06	5.63	-1.09
0.4	4.60	-1.24	6.06	-1.30
0.5	4.39	-1.34	5.82	-1.45
0.6	3.68	-1.39	4.96	-1.62
0.7	2.56	-1.46	3.55	-1.86
0.8	1.18	-1.57	1.62	-2.23
0.9	0.00	-2.40	0.00	-2.79
1.0	0.00	-4.90	0.00	-3.75

Exterior Wall Unfactored Live Load Shears (kip)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.11	-0.19	0.09	-0.16
0.1	0.11	-0.19	0.09	-0.16
0.2	0.11	-0.19	0.09	-0.16
0.3	0.11	-0.19	0.09	-0.16
0.4	0.11	-0.19	0.09	-0.16
0.5	0.11	-0.19	0.09	-0.16
0.6	0.11	-0.19	0.09	-0.16
0.7	0.11	-0.19	0.09	-0.16
0.8	0.11	-0.19	0.09	-0.16
0.9	0.11	-0.19	0.09	-0.16
1.0	0.11	-0.19	0.09	-0.16

Interior Wall Unfactored Live Load Shears (kip)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.23	-0.23	0.21	-0.21
0.1	0.23	-0.23	0.21	-0.21
0.2	0.23	-0.23	0.21	-0.21
0.3	0.23	-0.23	0.21	-0.21
0.4	0.23	-0.23	0.21	-0.21
0.5	0.23	-0.23	0.21	-0.21
0.6	0.23	-0.23	0.21	-0.21
0.7	0.23	-0.23	0.21	-0.21
0.8	0.23	-0.23	0.21	-0.21
0.9	0.23	-0.23	0.21	-0.21
1.0	0.23	-0.23	0.21	-0.21

Top Slab Unfactored Live Load Shears (kip)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	2.71	-0.26	3.24	-0.33
0.1	2.33	-0.33	2.67	-0.33
0.2	1.95	-0.47	2.11	-0.33
0.3	1.56	-0.69	1.59	-0.39
0.4	1.19	-1.00	1.14	-0.67
0.5	0.85	-1.37	0.78	-1.03
0.6	0.54	-1.74	0.49	-1.46
0.7	0.30	-2.10	0.27	-1.97
0.8	0.14	-2.44	0.12	-2.54
0.9	0.04	-2.76	0.04	-3.11
1.0	0.00	-3.05	0.00	-3.66

Bottom Slab Unfactored Live Load Shears (kip)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	2.19	-0.68	2.69	-0.68
0.1	1.61	-0.48	1.97	-0.48
0.2	1.06	-0.32	1.29	-0.32
0.3	0.54	-0.19	0.66	-0.21
0.4	0.06	-0.11	0.07	-0.14
0.5	0.01	-0.45	0.00	-0.46
0.6	0.02	-0.90	0.02	-0.96
0.7	0.02	-1.33	0.02	-1.40
0.8	0.01	-1.74	0.01	-1.80
0.9	0.00	-2.12	0.00	-2.15
1.0	0.00	-2.48	0.00	-2.46

E36-1.6.4 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 factors to loads on a 1-foot strip width of the box culvert. The minimum or maximum load factors may be used on each component to maximize the load effects. The results are as follows:

Strength 1 Moments

$$M_{str1} = \eta \cdot \left(\gamma st_{DC} \cdot M_{DC} + \gamma st_{DW} \cdot M_{DW} + \gamma st_{EV} \cdot M_{EV} + \gamma st_{EH} \cdot M_{EH} + \gamma st_{LS} \cdot M_{LS} + \gamma st_{LL} \cdot M_{LL}\right)$$

Corner Bars	$Mstr1_{CB} = 17.34$	kip-ft	(negative moment)
Positive Moment Top Slab Bars	$Mstr1_{PTS} = 19.59$	kip-ft	(positive moment)
Positive Moment Bottom Slab Bars	$Mstr1_{PBS} = 21.05$	kip-ft	(positive moment)
Negative Moment Top Slab Bars	$Mstr1_{NTS} = 22.00$	kip-ft	(negative moment)
Negative Moment Bottom Slab Bars	$Mstr1_{NBS} = 24.77$	kip-ft	(negative moment)
Exterior Wall Bars	$Mstr1_{XW} = 11.90$	kip-ft	(positive moment)
Interior Wall Bars	$Mstr1_{IW} = 4.82$	kip-ft	(positive moment)

Service 1 Moments

$$M_{s1} = \eta \cdot \left(\gamma s1_{DC} \cdot M_{DC} + \gamma s1_{DW} \cdot M_{DW} + \gamma s1_{EV} \cdot M_{EV} + \gamma s1_{EH} \cdot M_{EH} + \gamma s1_{LS} \cdot M_{LS} + \gamma s1_{LL} \cdot M_{LL}\right.$$

Corner Bars	$Ms1_{CB} = 11.18$	kip-ft	(negative moment)
Positive Moment Top Slab Bars	$Ms1_{PTS} = 11.66$	kip-ft	(positive moment)
Positive Moment Bottom Slab Bars	$Ms1_{PBS} = 12.32$	kip-ft	(positive moment)
Negative Moment Top Slab Bars	$Ms1_{NTS} = 13.15$	kip-ft	(negative moment)
Negative Moment Bottom Slab Bars	$Ms1_{NBS} = 15.08$	kip-ft	(negative moment)
Exterior Wall Bars	$Ms1_{XW} = 6.43$	kip-ft	(positive moment)
Interior Wall Bars	$Ms1_{IW} = 2.75$	kip-ft	(positive moment)

E36-1.7 Design Reinforcement Bars

Design of the corner bars is illustrated below. Calculations for bars in other locations are similar.

Design Criteria:

For corner bars, use the controlling thickness between the slab and wall. The height of the concrete design section is:

$h := \min(t_{ts}, t_{bs}, t_{wex})$ $\boxed{h = 12.00}$ in

Use a 1'-0" design width:

$b := 12.0$ width of the concrete design section, in

$cover = 2.0$ concrete cover, in Note: The calculations here use 2" cover for the top slab and walls. Use 3" cover for the bottom of the bottom slab (not shown here).

$Mstr1_{CB} = 17.34$ design strength moment, kip-ft

$Ms1_{CB} = 11.18$ design service moment, kip-ft

$f_s := f_y = 60.0$ reinforcement yield strength, ksi

$Bar_{No} := 5$ assume #5 bars (for d_s calculation)

$Bar_D(Bar_{No}) = 0.63$ bar diameter, in

Calculate the estimated distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement. **LRFD [5.7.3.2.2]**

$$d_{s\_i} := h - cover - \frac{Bar_D(Bar_{No})}{2}$$ $\boxed{d_{s\_i} = 9.69}$ in

For reinforced concrete cast-in-place box structures, $\phi_f = 0.90$ per **LRFD [Table 12.5.5-1]**.

Calculate the coefficient of resistance:

$$R_n := \frac{Mstr1_{CB} \cdot 12}{\phi_f \cdot b \cdot d_{s\_i}^{\;2}} = 0.21$$ $\boxed{R_n = 0.21}$ ksi

Calculate the reinforcement ratio:

$$\rho := 0.85 \cdot \frac{f'_c}{f_y} \cdot \left(1 - \sqrt{1.0 - \frac{2 \cdot R_n}{0.85 \cdot f'_c}}\right)$$ $\boxed{\rho = 0.0035}$

Calculate the required area of steel:

$A_{s\_req'd} := \rho \cdot b \cdot d_{s\_i}$ $\boxed{A_{s\_req'd} = 0.41}$ in^2

Given the required area of steel of $A_{s\_req'd} = 0.41$, try #5 bars at 7.5" spacing:

$Bar_{No} := 5$ bar size

$spacing := 7.5$ bar spacing, in

The area of one reinforcing bar is:

$A_{s\_1bar} := Bar_A(Bar_{No})$ $\boxed{A_{s\_1bar} = 0.31}$ in^2

Calculate the area of steel in a 1'-0" width

$$A_s := \frac{A_{s\_1bar}}{\frac{spacing}{12}}$$ $\boxed{A_s = 0.50}$ in^2

Check that the area of steel provided is larger than the required area of steel

Is $A_s = 0.50$ in^2 $\geq$ $A_{s\_req'd} = 0.41$ in^2 $\boxed{check = \text{"OK"}}$

Recalculate d_c and d_s based on the actual bar size used.

$$d_c := cover + \frac{Bar_D(Bar_{No})}{2}$$ $\boxed{d_c = 2.31}$ in

$$d_s := h - cover - \frac{Bar_D(Bar_{No})}{2}$$ $\boxed{d_s = 9.69}$ in

Per **LRFD [5.7.2.2]**, The factor β_1 shall be taken as 0.85 for concrete strengths not exceeding 4.0 ksi. For concrete strengths exceeding 4.0 ksi, β_1 shall be reduced at a rate of 0.05 for each 1.0 ksi of strength in excess of 4.0 ksi, except that β_1 shall not be taken to be less than 0.65.

$\boxed{\beta_1 = 0.85}$

Per **LRFD [5.7.2.1]**, if $\frac{c}{d_s} \leq 0.6$ then reinforcement has yielded and the assumption is correct.

"c" is defined as the distance between the neutral axis and the compression face (inches).

$$c := \frac{A_s \cdot f_s}{0.85 \cdot f'_c \cdot \beta_1 \cdot b}$$ $\boxed{c = 0.98}$ in

Check that the reinforcement will yield:

Is $\frac{c}{d_s} = 0.10 \leq 0.6?$ $\boxed{check = \text{"OK"}}$

therefore, the reinforcement will yield

Calculate the nominal moment capacity of the rectangular section in accordance with **LRFD [5.7.3.2.3]**:

$a := \beta_1 \cdot c$ $\boxed{a = 0.83}$ in

$M_n := \left[A_s \cdot f_s \cdot \left(d_s - \frac{a}{2}\right)\frac{1}{12}\right]$ $\boxed{M_n = 23.0}$ kip-ft

For reinforced concrete cast-in-place box structures, $\phi_f = 0.90$ **LRFD [Table 12.5.5-1]**.

Therefore the usable capacity is:

$M_r := \phi_f \cdot M_n$ $\boxed{M_r = 20.7}$ kip-ft

The required capacity:

Corner Moment $\boxed{Mstr1_{CB} = 17.3}$ kip-ft

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

$b = 12.0$	in	width of the concrete design section, in
$h = 12.0$	in	height of the concrete design section, in
$f_r := 0.37 \cdot \sqrt{f'_c} = 0.69$		modulus of rupture, ksi **LRFD [5.4.2.6]**
$I_g := \frac{1}{12} \cdot b \cdot h^3 = 1728$		gross moment of inertia, in^4
$\frac{h}{2} = 6.0$		distance from the neutral axis to the extreme element
$S_c := \frac{I_g}{\left(\frac{h}{2}\right)} = 288.00$		section modulus, in^3

The corresponding cracking moment is:

$M_{cr} := f_r \cdot S_c \cdot \frac{1}{12} = 16.61$ $\boxed{1.2 \cdot M_{cr} = 19.9}$ kip-ft

$\boxed{1.33 \cdot Mstr1_{CB} = 23.1}$ kip-ft

Is $M_r = 20.7$ kip-ft greater than the lesser of 1.2*M_{cr} and 1.33*M_{str}? $\boxed{check = \text{"OK"}}$

Per **LRFD [5.7.3.4]**, the spacing(s) of reinforcement in the layer closest to the tension face shall satisfy:

$$s \le \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$$ in which: $$\beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)}$$

$\gamma_e := 1.0$ 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:

$$N := \frac{E_s}{E_c} = 8.06 \qquad \boxed{N = 8.06}$$

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

$$f_{ss} := \frac{Ms1_{CB} \cdot 12}{A_s \cdot (j) \cdot (h - d_c)} \qquad \boxed{f_{ss} = 30.23} \text{ ksi}$$

Calculate the maximum spacing requirements per **LRFD [5.10.3.2]**:

$$s_{max1} := \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c \qquad s_{max1} = 12.64 \text{ in}$$

$$s_{max2} := \min(1.5 \cdot h, 18) \qquad s_{max2} = 18.00 \text{ in}$$

$$s_{max} := \min(s_{max1}, s_{max2}) \qquad \boxed{s_{max} = 12.64} \text{ in}$$

Check that the provided spacing is less than the maximum allowable spacing

Is $spacing = 7.50$ in $\leq$ $s_{max} = 12.64$ in $\boxed{check = \text{"OK"}}$

Calculate the minimum spacing requirements per **LRFD [5.10.3.1]**. The clear distance between parallel bars in a layer shall not be less than:

$S_{min1} := 1.5 \cdot Bar_D(Bar_{No}) = 0.94$ in

$S_{min2} := 1.5 \cdot 1.5 = 2.25$ in (maximum aggregate size = 1.5 inches)

$S_{min3} := 1.5$ in

Is $spacing = 7.50$ in $\geq$ all minimum spacing requirements? $\boxed{check = \text{"OK"}}$

E36-1.8 Shrinkage and Temperature Reinforcement Check

Check shrinkage and temperature reinforcement criteria for the reinforcement selected in preceding sections.

The area of reinforcement (A_s) per foot, for shrinkage and temperature effects, on each face and in each direction shall satisfy: **LRFD [5.10.8]**

$$A_s \geq \frac{1.30 \cdot b \cdot (h)}{2 \cdot (b + h) \cdot f_y} \quad \text{and} \quad 0.11 \leq A_s \leq 0.60$$

Where:

A_s = area of reinforcement in each direction and each face $\left(\frac{in^2}{ft}\right)$

b = least width of component section (in.)

h = least thickness of component section (in.)

f_y = specified yield strength of reinforcing bars (ksi) $\leq$ 75 ksi

Check the minimum required temperature and shrinkage reinforcement, #4 bars at 15", in the thickest section. For the given cross section, the values for the corner bar design are:

$A_{s\_4\_at\_15} := \frac{Bar_A(4)}{1.25}$ $\boxed{A_{s\_4\_at\_15} = 0.16}$ $\frac{in^2}{ft}$

$b_{TS} := \max(t_{ts}, t_{bs}, t_{wex})$ $\boxed{b_{TS} = 14.0}$ in

$h_{TS} := 12(W_1 + W_2) + 2 \cdot t_{wex} + t_{win} = 324.00$ $\boxed{h_{TS} = 324.0}$ in

$f_y = 60.00$ ksi

For each face, the required area of steel is:

$$A_{s\_TS} := \frac{1.30 \cdot (b_{TS}) \cdot h_{TS}}{2 \cdot (b_{TS} + h_{TS}) \cdot f_y} \qquad A_{s\_TS} = 0.15 \quad \frac{in^2}{ft}$$

is $A_{s\_4\_at\_15} = 0.16$ in2 $\geq$ $A_{s\_TS} = 0.15$ in^2 ? check = "OK"

is $0.11 < A_{s\_4\_at\_15} < 0.60$? check = "OK"

Per **LRFD [5.10.8]**, the shrinkage and temperature reinforcement shall not be spaced farther apart than:

- 3.0 times the component thickness, or 18.0 in.
- 12.0 in for walls and footings greater than 18.0 in. thick
- 12.0 in for other components greater than 36.0 in. thick

$s_{max3} = 18.00$ in

Per **LRFD [5.10.3.2]**, the maximum center to center spacing of adjacent bars shall not exceed 1.5 times the thickness of the member or 18.0 in.

$s_{max4} = 18.00$ in

is the 15" spacing $\leq$ both maximum spacing requirements? check = "OK"

The results for the other bar locations are shown in the table below:

Results						
Location	ΦMn	$A_{S\ Req'd}$	$A_{S\ Actual}$	Bar Size	S_{max}	S_{actual}
Corner	20.7	0.48	0.50	5	12.6	7.5
Pos. Mom. Top Slab	21.8	0.49	0.50	5	13.0	7.5
Pos. Mom. Bot. Slab	28.9	0.54	0.57	5	18.0	6.5
Neg. Mom. Top Slab	23.3	0.50	0.53	5	12.1	7.0
Neg. Mom. Bot. Slab	28.4	0.54	0.62	5	13.4	6.0
Exterior Wall	16.9	0.37	0.40	4	18.0	6.0
Interior Wall	6.9	0.15	0.16	4	18.0	15.0

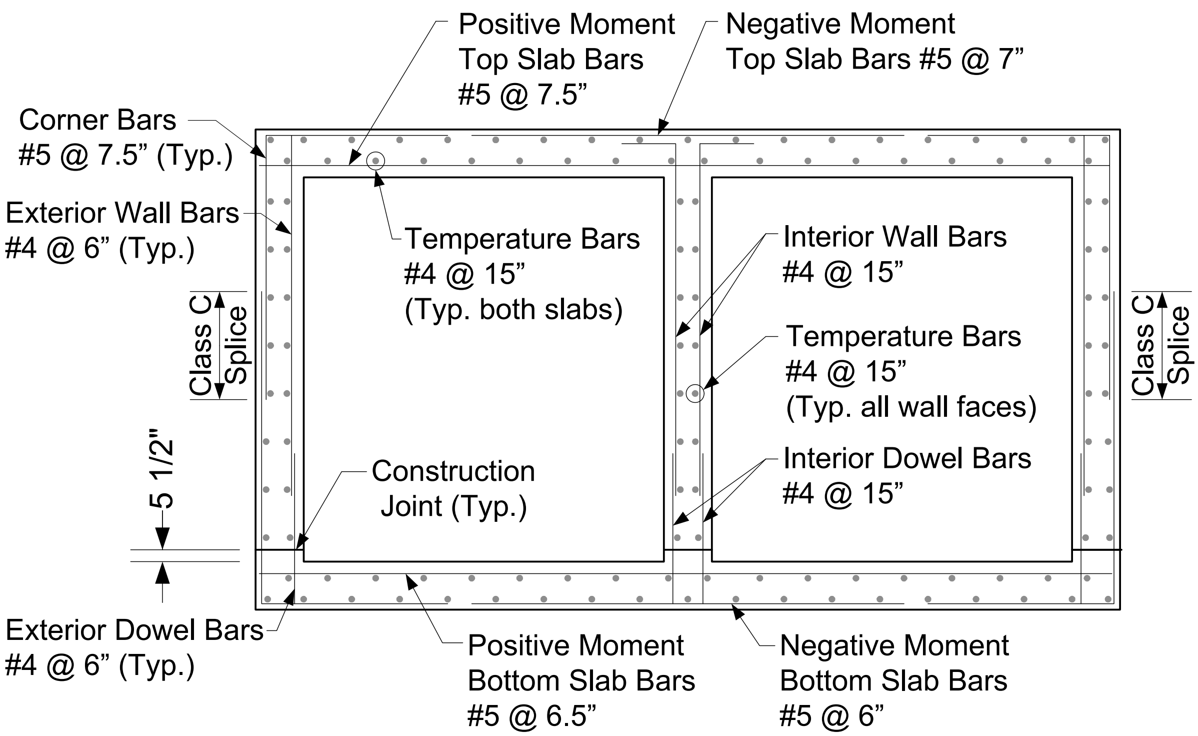
E36-1.9 Distribution Reinforcement

Per **LRFD [9.7.3.2]**, reinforcement shall be placed in the secondary direction in the bottom of slabs as a percentage of the primary reinforcement for positive moment as follows:

Distribution steel is not required when the depth of fill over the slab exceeds 2 feet, **LRFD [5.14.4.1]**.

E36-1.10 Reinforcement Details

The reinforcement bar size and spacing required from the strength and serviceability calcuations above are shown below:



E36-1.11 Cutoff Locations

Determine the cutoff locations for the corner bars. Per **[36.6.1]**, the distance "L" is computed from the maximum negative moment envelope for the top slab.

The cutoff lengths are in feet, measured from the inside face of the exterior wall.

Initial Cutoff Locations:

The initial cutoff locations are determined from the inflection points of the moment diagrams.

Corner Bars	$CutOff1_{CBH\_i} = 2.64$	$CutOff2_{CBH\_i} = 1.57$	Horizontal
		$CutOff2_{CBV\_i} = 2.37$	Vertical
Positive Moment Top Slab Bars	$CutOff1_{PTS\_i} = 1.26$	$CutOff2_{PTS\_i} = 1.86$	
Positive Moment Bottom Slab Bars	$CutOff1_{PBS\_i} = 1.27$	$CutOff2_{PBS\_i} = 1.97$	
Negative Moment Top Slab Bars	$CutOff1_{NTS\_i} = 8.63$	$CutOff2_{NTS\_i} = 10.32$	
Negative Moment Bottom Slab Bars	$CutOff1_{NBS\_i} = 8.97$	$CutOff2_{NBS\_i} = 10.56$	

For the second cutoff location for each component, the following checks shall be completed:

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

The required capacity at the second cutoff location (for the vertical leg of the corner bar):

$Mstr1_{CBV2} = 7.89$ strength moment at the second cutoff location, kip-ft

The usable capacity of the remaining bars is calculated as follows:

$$A_{s2} := \frac{A_s}{2} \qquad \boxed{A_{s2} = 0.25} \ \text{in}^2$$

$$c2 := \frac{A_{s2} \cdot f_s}{0.85 \cdot f'_c \cdot \beta_1 \cdot b} \qquad \boxed{c2 = 0.49} \ \text{in}$$

$$a2 := \beta_1 \cdot c2 \qquad \boxed{a2 = 0.42} \ \text{in}$$

$$M_{n2} := \left[A_{s2} \cdot f_s \cdot \left(d_s - \frac{a2}{2}\right) \frac{1}{12}\right] \qquad \boxed{M_{n2} = 11.8} \ \text{kip-ft}$$

$$M_{r2} := \phi_f \cdot M_{n2} \qquad \boxed{M_{r2} = 10.6} \ \text{kip-ft}$$

Is $M_{r2} = 10.6$ kip-ft greater than the lesser of 1.2*M_{cr} and 1.33*M_{str}? $\boxed{check = \text{"OK"}}$

$\boxed{1.2 \cdot M_{cr} = 19.9}$ 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]**:

$$MaxDepth := \max\left(t_{ts} - cover, t_{wex} - cover, t_{bs} - cover_{bot}\right) = 11.00 \quad \text{in}$$

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:

$$ExtendLength\_gen_{CB} = 0.89 \quad \text{ft}$$

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:

$$ExtendLength\_neg_{CB} = 0.89 \quad \text{ft}$$

The development length:

$$DevLength_{CB} = 1.00 \quad \text{ft}$$

The extension lengths for general reinforcment for the other bars are:

Corner Bars	$ExtendLength\_gen_{CB} = 0.89$	ft
Positive Moment Top Slab Bars	$ExtendLength\_gen_{PTS} = 0.85$	ft
Positive Moment Bottom Slab Bars	$ExtendLength\_gen_{PBS} = 0.97$	ft
Negative Moment Top Slab Bars	$ExtendLength\_gen_{NTS} = 0.85$	ft
Negative Moment Bottom Slab Bars	$ExtendLength\_gen_{NBS} = 0.97$	ft

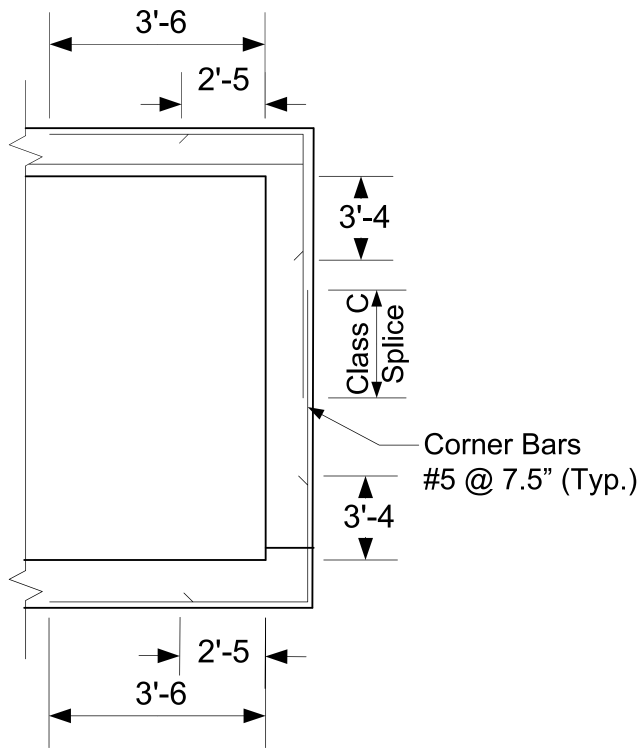
The extension lengths for negative moment reinforcment for the other bars are:

Corner Bars	$ExtendLength\_neg_{CB} = 0.89$	ft
Positive Moment Top Slab Bars	$ExtendLength\_neg_{PTS} = 0.85$	ft
Positive Moment Bottom Slab Bars	$ExtendLength\_neg_{PBS} = 0.97$	ft
Negative Moment Top Slab Bars	$ExtendLength\_neg_{NTS} = 0.85$	ft
Negative Moment Bottom Slab Bars	$ExtendLength\_neg_{NBS} = 0.97$	ft

The final cutoff locations (measured from the inside face of the exterior wall) are:

Corner Bars	$CutOff1_{CBH}$ = 3.53	$CutOff2_{CBH}$ = 2.46	Horizontal
		$CutOff2_{CBV}$ = 3.26	Vertical
Positive Moment Top Slab Bars	$CutOff1_{PTS}$ = "Run Bar Entire Width of Box"		
		$CutOff2_{PTS}$ = 1.02	
Positive Moment Bottom Slab Bars	$CutOff1_{PBS}$ = "Run Bar Entire Width of Box"		
		$CutOff2_{PBS}$ = 1.00	
Negative Moment Top Slab Bars	$CutOff1_{NTS}$ = 7.78	$CutOff2_{NTS}$ = 9.47	
Negative Moment Bottom Slab Bars	$CutOff1_{NBS}$ = 7.99	$CutOff2_{NBS}$ = 9.59	

The cutoff locations for the corner bars are shown below. Other bars are similar.



E36-1.12 Shear Analysis

E36-1.12.1 Factored Shears

WisDOT's policy is to set all of the load modifiers, η, equal to 1.0. The factored shears for each limit state are calculated by applying the appropriate load factors to loads on a 1-foot strip width of the box culvert. The minimum or maximum load factors may be used on each component to maximize the load effects. The results are as follows:

Strength 1 Shears

$$V_{str1} = \eta \cdot \left(\gamma st_{DC} \cdot V_{DC} + \gamma st_{DW} \cdot V_{DW} + \gamma st_{EV} \cdot V_{EV} + \gamma st_{EH} \cdot V_{EH} + \gamma st_{LS} \cdot V_{LS} + \gamma st_{LL} \cdot V_{LL}\right)$$

Exterior Wall	$Vstr1_{XW} = 8.69$	kip
Interior Wall	$Vstr1_{IW} = 0.40$	kip
Top Slab	$Vstr1_{TS} = 12.20$	kip
Bottom Slab	$Vstr1_{BS} = 12.16$	kip

Service 1 Shears

$$V_{s1} = \eta \cdot \left(\gamma s1_{DC} \cdot V_{DC} + \gamma s1_{DW} \cdot V_{DW} + \gamma s1_{EV} \cdot V_{EV} + \gamma s1_{EH} \cdot V_{EH} + \gamma s1_{LS} \cdot V_{LS} + \gamma s1_{LL} \cdot V_{LL}\right)$$

Exterior Wall	$Vs1_{XW} = 5.64$	kip
Interior Wall	$Vs1_{IW} = 0.23$	kip
Top Slab	$Vs1_{TS} = 7.62$	kip
Bottom Slab	$Vs1_{BS} = 7.96$	kip

E36-1.12.2 Concrete Shear Resistance

Check that the nominal shear resistance, V_n, of the concrete in the top slab is adequate for shear without shear reinforcement per **LRFD [5.14.5.3]**.

$$V_n = V_c = \left(0.0676 \cdot \sqrt{f'_c} + 4.6 \cdot \frac{A_s}{b \cdot d_s} \cdot \frac{V_u \cdot d_s}{M_u}\right) \cdot b \cdot d_s \le 0.126 \cdot \sqrt{f'_c} \cdot b \cdot d_s$$

$f'_c = 3.5$	culvert concrete strength, ksi
$A_{s\_TS} = 0.53$	area of reinforcing steel in the design width, in^2/ft width
$h := t_{ts} = 12.5$	height of concrete design section, in

Calculate d_s, the distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement:

$$d_s := h - cover - \frac{Bar_D(Bar_{No})}{2} \qquad \boxed{d_s = 10.19} \text{ in}$$

$$V_u := Vstr1_{TS} \qquad \boxed{V_u = 12.2} \text{ kips}$$

$M_u = 264.01$ factored moment occurring simultaneously with V_u, kip-in

$b := 12$ design width, in

For reinforced concrete cast-in-place box structures, $\phi_v = 0.85$, **LRFD [Table 12.5.5-1]**. Therefore the usable capacity is:

$\frac{V_u \cdot d_s}{M_u}$ shall not be taken to be greater than 1.0 $\frac{V_u \cdot d_s}{M_u} = 0.47$ < 1.0 OK

$$V_{r1s} := \phi_v \cdot \left[\left(0.0676 \cdot \sqrt{f'_c} + 4.6 \cdot \frac{A_{s\_TS}}{b \cdot d_s} \cdot \frac{V_u \cdot d_s}{M_u}\right) \cdot b \cdot d_s\right] \qquad \boxed{V_{r1s} = 14.1} \text{ kips}$$

but $\leq$ $V_{r2s} := \phi_v \cdot \left(0.126 \cdot \sqrt{f'_c} \cdot b \cdot d_s\right)$ $\boxed{V_{r2s} = 24.5}$ kips

$V_{rs} := \min(V_{r1s}, V_{r2s}) = 14.1$ $\boxed{V_{rs} = 14.1}$ kips

Check that the provided shear capacity is adequate:

Is $V_u = 12.2$ kip $\leq$ $V_{rs} = 14.1$ kip ? $\boxed{check = \text{"OK"}}$

Note: For single-cell box culverts only, V_c for slabs monolithic with walls need not be taken to be less than: $0.0948 \cdot \sqrt{f'_c} \cdot b \cdot d_s$

V_c for slabs simply supported need not be taken to be less than: $0.0791 \cdot \sqrt{f'_c} \cdot b \cdot d_s$

LRFD [5.8] and **LRFD [5.13.3.6]** apply to slabs of box culverts with less than 2.0 ft of fill.

Check that the nominal shear resistance, V_n, of the concrete in the walls is adequate for shear without shear reinforcement per **LRFD [5.8.3.3]**. Calculations shown are for the exterior wall.

$$V_n = V_c = 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v \leq 0.25 \cdot f'_c \cdot b_v \cdot d_v$$

$\beta := 2$ **LRFD [5.8.3.4.1]**

$f'_c = 3.5$ culvert concrete strength, ksi

$b_v := 12$ effective width, in

$h := t_{wex} = 12.0$ height of concrete design section, in

Distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement:

$$d_s := h - cover - \frac{Bar_D(Bar_{No})}{2} \qquad \boxed{d_s = 9.69} \text{ in}$$

The effective shear depth taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure; **LRFD [5.8.2.9]**

$$d_{v\_i} = d_s - \frac{c}{2}$$

from earlier calculations:

$$\boxed{\beta_1 = 0.85}$$

$$\boxed{f_s = 60} \text{ ksi}$$

$$\boxed{A_{s\_XW} = 0.40} \text{ in}^2$$

The distance between the neutral axis and the compression face:

$$c := \frac{A_{s\_XW} \cdot f_s}{0.85 \cdot f'_c \cdot \beta_1 \cdot b_v} \qquad \boxed{c = 0.79} \text{ in}$$

The effective shear depth:

$$d_{v\_i} := \left(d_s - \frac{c}{2}\right) \qquad \boxed{d_{v\_i} = 9.29}$$

d_v need not be taken to be less than the greater of 0.9 d_s or 0.72h (in.)

$$d_v := \max\left(d_{v\_i}, \max\left(0.9d_s, 0.72t_{wex}\right)\right) = 9.29 \text{ in} \qquad 0.9 \cdot d_s = 8.72$$

$$0.72 \cdot t_{wex} = 8.64$$

For reinforced concrete cast-in-place box structures, $\phi_v = 0.85$, **LRFD [Table 12.5.5-1]**.

Therefore the usable capacity is:

$$V_{r1w} := \phi_v \cdot \left(0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v\right) \qquad \boxed{V_{r1w} = 11} \text{ kips}$$

$$\text{but} \le \quad V_{r2w} := \phi_v \cdot \left(0.25 \cdot f'_c \cdot b_v \cdot d_v\right) \qquad \boxed{V_{r2w} = 83} \text{ kips}$$

$$V_{rw} := \min\left(V_{r1w}, V_{r2w}\right) = 11 \qquad \boxed{V_{rw} = 11} \text{ kips}$$

$$V_u := Vstr1_{XW} \qquad \boxed{V_u = 8.7} \text{ kips}$$

Check that the provided shear capacity is adequate:

Is $V_u = 8.7$ kip $\le$ $V_{rw} = 11.2$ kip ? $\boxed{check = \text{"OK"}}$

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45.8 Load Rating Documentation

45.8.1 Load Rating Summary Sheet

After the structure has been load rated, the WisDOT Bridge Load Rating Summary Form shall be completed and utilized as a cover sheet for the load rating calculations (see Figure 45.8-1). This form may be obtained from the Bureau of Structures or is available on the following website:

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

Note: The Load Rating Summary Form is not required to be completed and sent in for concrete box culvert structures.

Instructions for completing the form are as follows:

1. Check what method was used to rate the bridge in the space provided.

2. Enter all data for all items corresponding to the vehicle type. Capacities for the posting vehicles do not have to be calculated if the Operating rating factor is greater than 1.0 for the HL-93 (LRFR) or the HS20 (LFR).

3. The rating for the Wis-SPV is always required and should be given on the rating sheet for both a multi-lane distribution and a single-lane distribution. Make sure not to include the future wearing surface in these calculations. All reported ratings are based on current conditions and do not reflect future wearing surfaces.

4. For the Operating rating, enter the lowest rating for each appropriate vehicle type, subject to the above requirements.

5. For the controlling element, make sure to enter the element (Slab, deck girder, lower truss chord, etc.) as well as the check (moment, shear, etc).

6. Be specific in describing where the controlling rating is located. For example, for girder bridges, enter the controlling span, girder-line, and location within the span (Ex. Span 2, G3, midspan).

7. For the live load distribution factor, enter the DF for the controlling element. Be sure to specify if it is a shear DF or a moment DF.

8. Enter all additional remarks as required to clarify the load capacity calculations and, if necessary, recommend posting signage.

9. It is necessary for the responsible engineer to sign and seal the form in the space provided.

45.8.2 Load Rating on Plans

The plans shall contain the following rating information:

- Inventory Load Rating – The plans shall have either the HS value of the inventory rating if using LFR or the rating factor for the HL-93 if using LRFR. This rating shall be based on the current conditions of the bridge at the point when the construction is complete and shall not use the future wearing surface. For new concrete box culvert structures, place a rating factor of 1.05 on the plans.

- Operating Load Rating – The plans shall have either the HS value of the operating rating if using LFR or the rating factor for the HL-93 if using LRFR. This rating shall be based on the current conditions of the bridge at the point when the construction is complete and shall not use the future wearing surface. For new concrete box culvert structures, place a rating factor of 1.35 on the plans.

- Wis-SPV – The plans shall also contain the results of the Wis-SPV analysis utilizing single-lane (single trip permit) distribution and assuming that the vehicle is mixing with normal traffic and that the full dynamic load allowance is utilized. This rating shall be based on the current conditions of the bridge at the point when the construction is complete and shall not use the future wearing surface. The recorded rating for this is the total allowable vehicle weight rounded down to the nearest 10 kips. If the value exceeds 250 kips, limit the plan value to 250 kips. For new concrete box culvert structures, place a value of 255 kips for the allowable vehicle weight on the plans.

Note: The culvert ratings indicated above will be used by BOS as a placeholder until policy (AASHTO and WisDOT) is determined for rating culverts.