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2.1 Organizational Charts

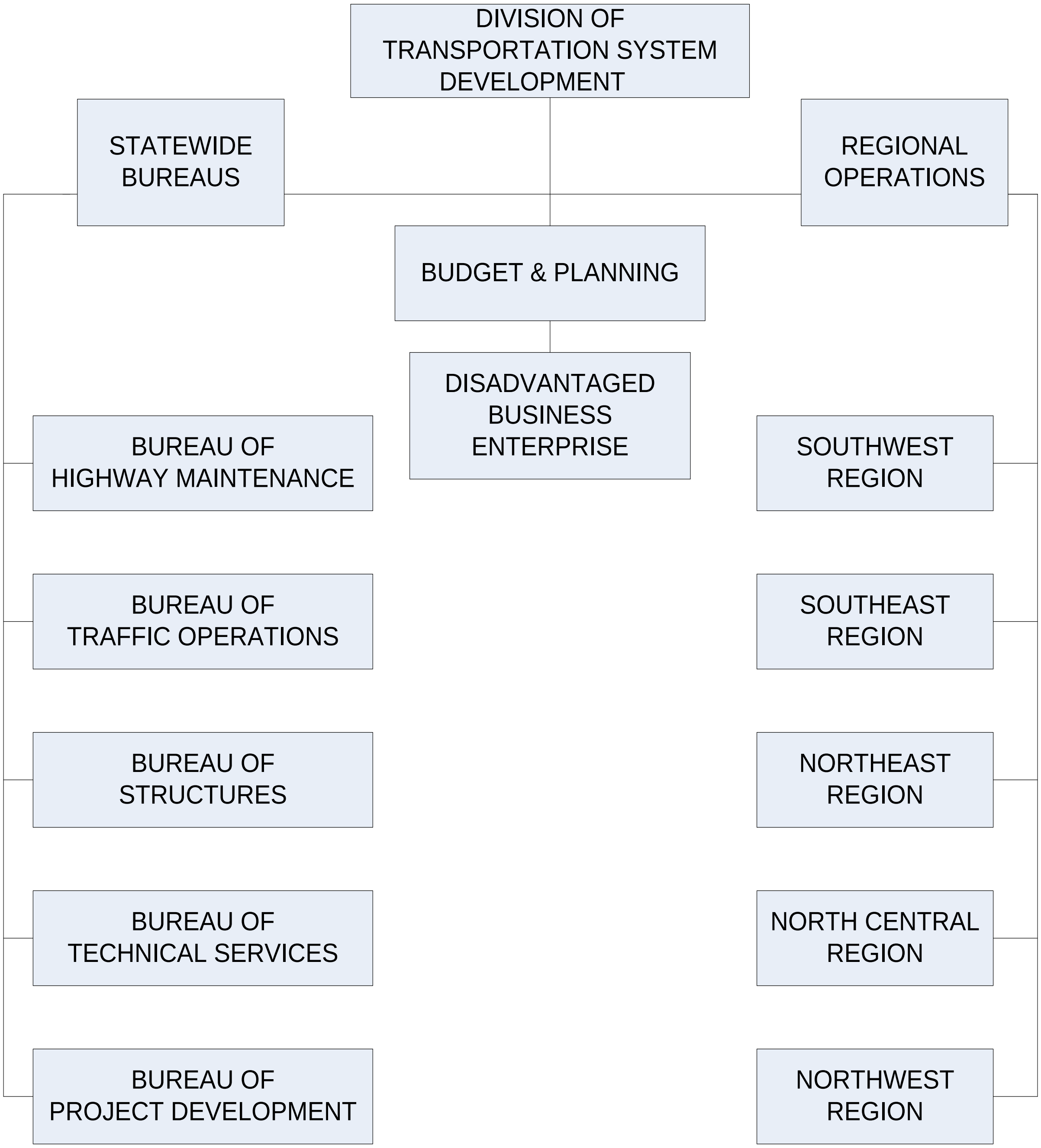

Figure 2.1-1
Division of Transportation System Development

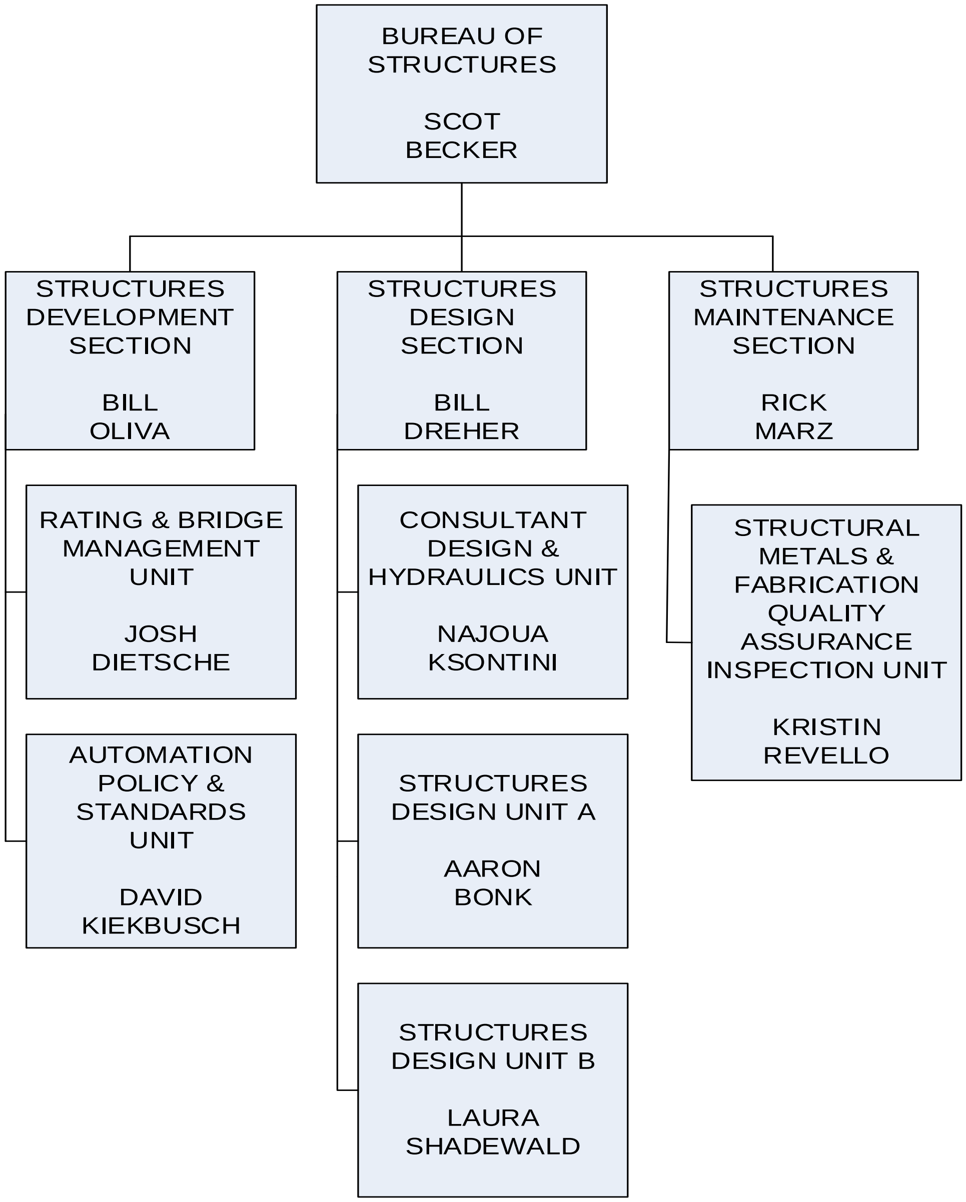

Figure 2.1-2
Bureau of Structures

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

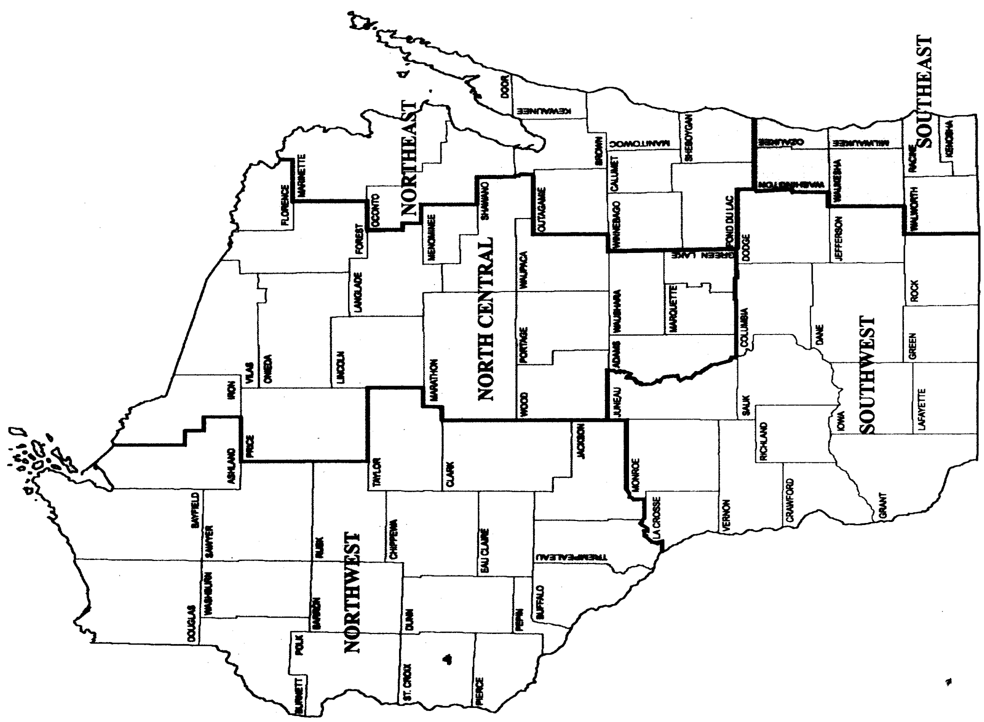

Figure 2.1-3
Region Map

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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.

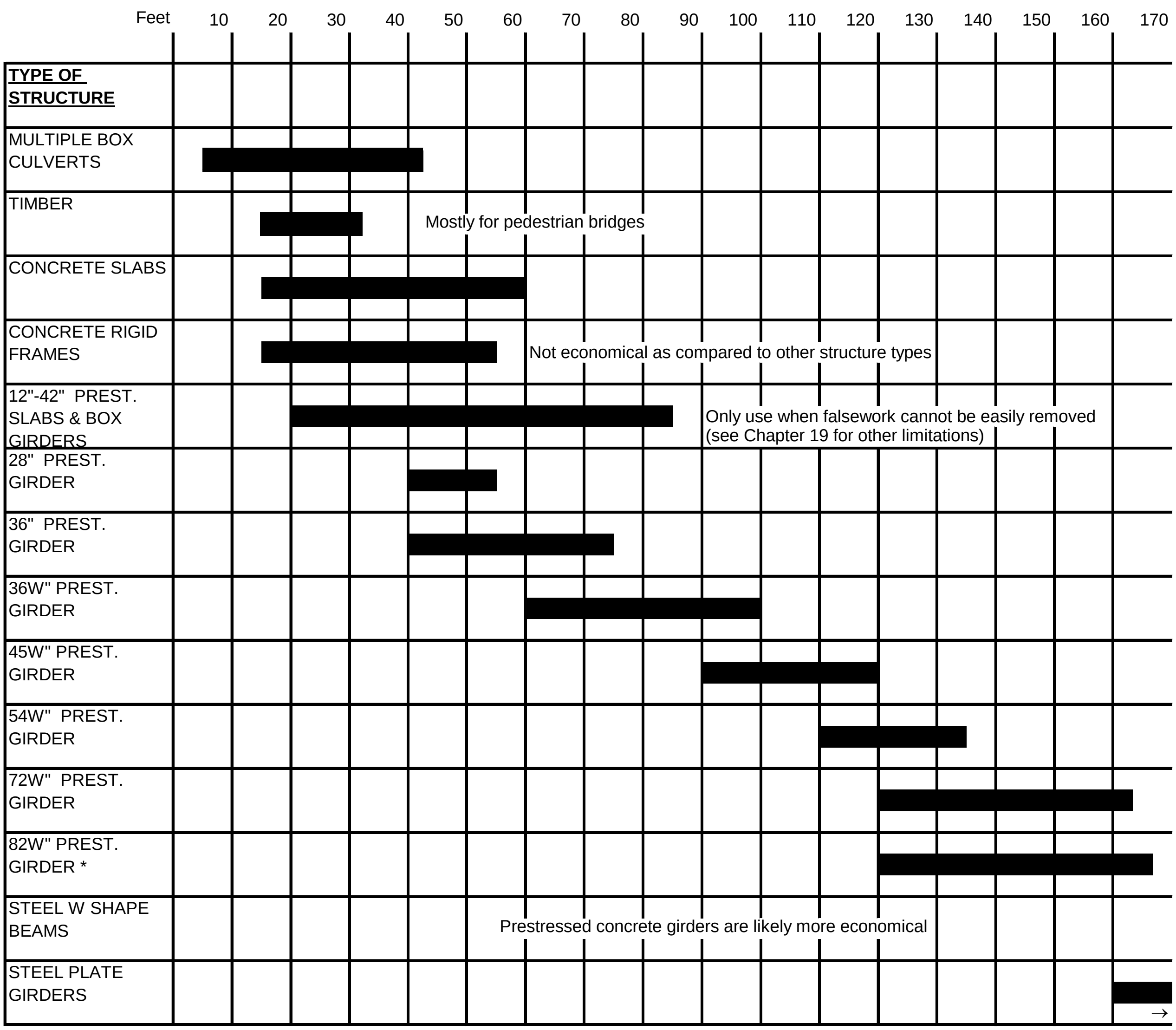
In these reports prestressed girder costs are grouped together because there is a small cost difference between girder sizes. Refer to unit costs. Concrete slab costs are also grouped together for this reason.

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.

Other available estimating tools such as *AASHTOWare Project Estimator* and *Bid Express*, as described in FDM 19-5-5, should be the primary tools for structure project cost estimations. Information in this chapter can be used as a supplemental tool.

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
502.3100	Expansion Device (structure) (LS)	LF	$206.63
502.3110.S	Expansion Device Modular (structure) (LS)	LF	$1401.52
SPV.0105	Expansion Device Modular LRFD (structure) (LS)	LF	$1947.75

Table 5.3-1
Contract Unit Bid Prices for Structures - 2017

Other bid items should be looked up in Estimator or Bid Express

5.4Bid 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 2013 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	17	120,700	12,295,720	49.75	101.87
Reinf. Conc. Slabs (All but A5)	12	26,361	2,244,395	48.26	85.14
Reinf. Conc. Slabs (A5 Abuts)	5	8,899	992,966	49.28	111.58

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	52	672,482	67,865,859	69.67	100.92
Steel Plate Girders	6	195,462	27,809,905	89.62	142.28
Trapezoidal Steel Box Girders	7	571,326	98,535,301	116.21	172.47

Table 5.4-7
Grade Separation Structures

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

Table 5.4-8
Box Culverts

Bridge Type	Cost per Sq. Ft.
Pre-Fab Pedestrian Bridge (B-13-661)	222.06
Pre-Fab Pedestrian Bridge (B-13-666)	240.30
Pre-Fab Pedestrian Bridge (B-17-211)	174.33
Pre-Fab Pedestrian Bridge (B-40-784)	289.02
Concrete Slab Pedestrian Bridge (B-13-656)	105.60
Concrete Slab Pedestrian Bridge (B-13-657)	106.62
Buried Slab Bridge (B-24-40)	182.28
Buried Slab Bridge (B-5-403)	165.57
Buried Slab Bridge (B-13-654)	210.68
Railroad Bridge (B-40-773)	1,151.00
Railroad Bridge (B-40-774)	1,541.00
Inverted T Bridge (B-13-608)	192.75
Inverted T Bridge (B-13-609)	235.01
Inverted T Bridge (B-40-89)	528.81

Table 5.4-9
Miscellaneous Bridges

Retaining Wall Type	No. of Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Block Walls	8	13,351	447,017	33.48
MSE Panel Walls	55	255,817	23,968,072	93.69
Concrete Walls	23	32,714	2,991,867	91.46
Panel Walls	7	39,495	8,028,652	203.28
Wire Faced MSE Walls	28	160,296	20,554,507	128.17

Table 5.4-10
Retaining Walls

5.4.2 2014 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	457,537	52,424,589	53.80	114.58
Reinf. Conc. Slabs (All but A5)	27	59,522	8,104,551	58.89	136.16
Reinf. Conc. Slabs (A5 Abuts)	9	16,909	2,150,609	56.13	127.19
Buried Slab Bridges	1	4,020	198,583	11.63	49.40

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	29	409,929	44,335,036	64.66	108.15
Reinf. Conc. Slabs (All but A5)	2	15,072	1,739,440	47.68	115.41
Steel Plate Girders	3	85,715	15,669,789	114.08	182.81
Steel I-Beams	1	2,078	596,712	82.99	287.16
Trapezoidal Steel Box Girders	1	59,128	9,007,289	121.00	152.34
Pedestrian Bridges	3	35,591	7,436,429	--	208.94

Table 5.4-12
Grade Separation Structures

Box Culvert Type	No. of Culverts	Cost per Lin. Ft.
Single Cell	10	2,361.30
Twin Cell	4	2,584.21
Triple Cell	1	2,928.40
Triple Pipe	1	1,539.41

Table 5.4-13
Box Culverts

Retaining Wall Type	No. of Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Block Walls	11	13,856	755,911	54.55
MSE Panel Walls	36	319,463	23,964,444	75.01
Concrete Walls	7	58,238	8,604,747	147.75
Panel Walls	1	3,640	590,682	162.28
Wire Faced MSE Walls	2	3,747	537,173	143.36
Secant Pile Walls	1	68,326	7,488,658	109.60
Soldier Pile Walls	9	33,927	4,470,908	131.78
Steel Sheet Pile Walls	2	3,495	159,798	45.72

Table 5.4-14
Retaining Walls

Noise Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
13	200,750	5,542,533	27.61

Table 5.4-15
Noise Walls

5.4.3 2015 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	22	338,229	41,220,154	60.96	121.87
Reinf. Conc. Slabs (Flat)	26	47,766	7,151,136	62.77	149.71
Reinf. Conc. Slabs (Haunched)	6	27,967	3,517,913	57.49	125.79
Buried Slab Bridges	1	2,610	401,000	43.74	153.64
Pre-Fab Pedestrian Bridges	3	29,304	3,440,091	--	117.39

Table 5.4-16
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	58	768,458	102,067,913	66.04	132.82
Reinf. Conc. Slabs (Flat)	2	8,566	922,866	46.36	107.74
Reinf. Conc. Slabs (Haunched)	1	6,484	868,845	41.26	133.99
Steel Plate Girders	4	100,589	20,248,653	137.13	201.30
Trapezoidal Steel Box Girders	4	305,812	79,580,033	189.24	260.23
Rigid Frames	2	7,657	2,730,308	--	356.58
Timber	1	16,800	1,982,669	--	118.02
Pre-Fab Pedestrian Bridges	1	1,851	449,475	--	242.83

Table 5.4-17
Grade Separation Structures

Box Culvert Type	No. of Culverts	Cost per Lin. Ft.
Single Cell	2	2,235.67
Twin Cell	6	3,913.05
Single Pipe	1	2,262.11
Double Pipe	2	426.20
Triple Pipe	2	1,424.09
Quadruple Pipe	1	2,332.96

Table 5.4-18
Box Culverts

Retaining Wall Type	No. of Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Block Walls	11	22,353	1,594,171	71.32
MSE Panel Walls	51	315,440	28,038,238	88.89
MSE Panel Walls w/Integral Barrier	4	14,330	1,098,649	76.67
Concrete Walls	2	6,850	712,085	103.96
Wire Faced MSE Walls	3	10,345	1,501,948	145.19
Wire Faced MSE Walls w/ Precast Conc. Wall Panels	12	50,670	10,195,161	201.21

Secant Pile Walls	1	5,796.50	1,075,785	185.59
Soldier Pile Walls	6	37,498	6,037,788	161.02
Steel Sheet Pile Walls	6	11,319	668,227	59.04

Table 5.4-19
Retaining Walls

Sign Structure Type		No. of Structures	Total Lineal Ft. of Arm	Total Costs	Cost per Lin. Ft.
Butterfly (1-Sign)	Conc. Col.	2	44	122,565	2,785.56
	1-Steel Col.	2	42	63,965	1,522.98
Butterfly (2-Signs)	1-Steel Col.	1	21	48,971	2,331.97
Cantilever	Conc. Col	18	530	1,217,454	2,297.08
	1-Steel Col.	15	394	528,950	1,342.85
Full Span	Conc. Col.	44	4,035	5,309,906	1,315.96
	1-Steel Col.	12	720	476,598	662.00
	2-Steel Col.	10	711	775,858	1,091.22
Full Span + Cantilever	Conc. Col.	1	84	166,003	1,976.22

Table 5.4-20
Sign Structures

5.4.4 2016 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	19	199,367	26,157,660	57.97	131.20
Reinf. Conc. Slabs (Flat)	36	72,066	10,985,072	63.40	152.43
Reinf. Conc. Slabs (Haunched)	5	22,144	2,469,770	50.63	111.53
Prestressed Box Girders	3	4,550	773,098	80.85	169.91

Table 5.4-21
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	25	343,165	40,412,805	60.62	117.76
Reinf. Conc. Slabs (Haunched)	5	33,268	4,609,286	59.21	138.55
Steel Plate Girders	3	127,080	18,691,714	90.78	147.09
Pedestrian Bridges	1	4,049	846,735	91.35	209.13

Table 5.4-22
Grade Separation Structures

Box Culvert Type	No. of Culverts	Cost per Lin. Ft.
Single Cell	18	1,694.52
Twin Cell	10	2,850.45
Single Pipe	1	1,268.42

Table 5.4-23
Box Culverts

Retaining Wall Type	No. of Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Block Walls	10	10,310	558,347	54.16
MSE Panel Walls	21	112,015	8,681,269	77.50
Modular Walls	5	6,578	419,334	63.75
Soldier Pile Walls	2	13,970	1,208,100	86.48
Steel Sheet Pile Walls	1	3,440	104,814	30.47

Table 5.4-24
Retaining Walls

Sign Structure Type		No. of Structures	Total Lineal Ft. of Arm	Total Costs	Cost per Lin. Ft.
Butterfly (2-Signs)	Conc. Col.	1	25.25	89,102	3,528.80
	1-Steel Col.	1	24.34	44,176	1,814.97
Cantilever	Conc. Col	5	171	384,487	2,248.46
	1-Steel Col.	18	536.25	758,646	1,414.72
Full Span	Conc. Col.	0	--	--	--
	1-Steel Col.	7	430.25	400,125	929.98
	2-Steel Col.	7	590	611,292	1,036.23

Table 5.4-25
Sign Structures

5.4.5 2017 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	238,956	33,970,344.86	60.05	142.16
Reinf. Conc. Slabs (Flat)	44	69,095	11,063,299.53	57.75	160.12
Reinf. Conc. Slabs (Haunched)	8	48,434	6,759,897.64	55.41	139.57
Prestressed Box Girders	2	2,530	691,474.35	117.93	273.31

Table 5.4-21
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	28	302,672	37,247,580.94	52.67	123.10
Reinf. Conc. Slabs (Flat)	25	58,076	9,561,823.06	42.14	164.64
Reinf. Conc. Slabs (Haunched)	6	49,160	9,444,012.75	43.73	192.11
Steel Plate Girders	0	--	--	--	--
Pedestrian Bridges	2	12,864	2,141,133.01	53.53	166.44

Table 5.4-22
Grade Separation Structures

Box Culvert Type	No. of Culverts	Cost per Lin. Ft.
Single Cell	18	1,849.26
Twin Cell	3	3,333.61
Single Pipe	1	1,752.93
Precast	1	2,204.32
Precast Three-Sided	3	8,754.76

Table 5.4-23
Box Culverts

Retaining Wall Type	No. of Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
CIP Cantilever	17	30,808	3,277,766.33	106.39
CIP Facing (MSE)	3	10,611	1,683,447.67	158.65
MSE Block Walls	6	13,378	1,457,896.15	108.98
MSE Panel Walls	21	137,718	11,789,074.54	85.60
Modular Walls	3	3,643	254,004.30	69.72
Precast Panel and Wire Faced	3	17,270	2,294,507.57	132.86
Soldier Pile Walls	0	--	--	--
Steel Sheet Pile Walls	5	15,056	1,442,741.15	95.82

Table 5.4-24
Retaining Walls

Sign Structure Type		No. of Structures	Total Lineal Ft. of Arm	Total Costs	Cost per Lin. Ft.
Butterfly (1-Sign)	Conc. Col.	0	--	--	--
	1-Steel Col.	4	84.5	221,728.47	2,623.01
Butterfly (2-Signs)	Conc. Col.	0	--	--	--
	1-Steel Col.	6	217.22	417,307.35	1,921.13
Cantilever	Conc. Col	0	--	--	--
Cantilever Full Span	1-Steel Col.	28	825.75	1,165,570.03	1,411.53
	2-Steel Col.	2	199	245,997.03	1,236.17
	Conc. Col.	2	185	349,166.59	1887.39
Full Span	1-Steel Col.	6	466.03	589,773.11	1265.53
	2-Steel Col.	21	1,773.5	1,789,041.14	1008.76

Table 5.4-25
Sign Structures

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6.3 Final Plans

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

6.3.1 General Requirements

6.3.1.1 Drawing Size

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

6.3.1.2 Scale

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

6.3.1.3 Line Thickness

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

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

6.3.1.4 Lettering and Dimensions

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

6.3.1.5 Notes

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

6.3.1.6 Standard Insert Drawings

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

Standard insert sheets can be found at: https://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/strct/insert-sheets.aspx

6.3.1.7 Abbreviations

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

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

Reference	REF.	Station	STA.
Reinforcement	REINF.	Structural	STR.
Reinforced Concrete Culvert Pipe	R.C.C.P.	Substructure	SUBST.
Required	REQ'D.	Superstructure	SUPER.
Right	RT.	Surface	SURF.
Right Hand Forward	R.H.F.	Superelevation	S.E.
Right of Way	R/W	Symmetrical	SYM
Roadway	RDWY.	Tangent Line	TAN. LN.
Round	ø	Transit Line	T/L
Section	SEC.	Transverse	TRAN.
Shoulder	SHLD.	Variable	VAR.
Sidewalk	SDWK.	Vertical	VERT.
South	S.	Vertical Curve	V.C.
Space	SPA.	Volume	VOL.
Specification	SPEC	West	W.
Standard	STD.	Zinc Gauge	ZN. GA.

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

1. Plan View

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

   a. Sufficient dimensions to layout structure in the field.

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

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

   On Structure Replacements

   Show existing structure in dashed-lines on Plan View.

2. Elevation View

   Same requirements as specified for preliminary plan except:

   a. Show elevation at bottom of all substructure units.

   b. Give estimated pile lengths where used.

3. Cross-Section View

   Same requirements as specified for preliminary plan except:

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

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

4. Grade Line

   Same requirements as specified for preliminary plan.

5. Design and Traffic Data

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

6. Hydraulic Information, if Applicable

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

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

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

Record this quantity to the nearest 1 cubic yard.

6.4.15 Pile Points

When recommended in soils report. Bid as each.

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

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

6.4.17 Cofferdams (Structure)

Lump Sum

6.4.18 Rubberized Membrane Waterproofing

Record the quantity to the nearest square yard.

6.4.19 Expansion Devices

For “Expansion Device” and “Expansion Device Modular”, bid the items in lineal feet. The distance measured is from flowline to flowline along the skew (do not include turn-ups into parapets or medians).

6.4.20 Electrical Work

Refer to Standard Construction Specifications for bid items.

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

Record this quantity in feet.

6.4.22 Preparation Decks Type 1 or Preparation Decks Type 2

Record these quantities to the nearest square yard. Preparation Decks Type 1 should be provided by the Region. Estimate Preparation Decks Type 2 as 40% of Preparation Decks Type 1. Deck preparation areas shall be filled using Concrete Masonry Overlay Decks, Concrete Masonry Deck Repair, or with an appropriate deck patch. See Chapter 40 Standards.

6.4.23 Cleaning Decks

Record this quantity to the nearest square yard.

6.4.24 Joint Repair

Record this quantity to the nearest square yard.

6.4.25 Concrete Surface Repair

Record this quantity to the nearest square foot.

6.4.26 Full-Depth Deck Repair

Record this quantity to the nearest square yard.

6.4.27 Concrete Masonry Overlay Decks

Record this quantity to the nearest cubic yard. Estimate the quantity by using a thickness measured from the existing ground concrete surface to the plan gradeline. Calculate the minimum overlay thickness and add ½" for variations in the deck surface. Provide this average thickness on the plan, as well. Usually 1" of deck surface is removed by grinding. Include deck repair quantities for Preparation Decks Type 1 & 2 and Full-Depth Deck Repair. Use 2-inch thickness for each Preparation area and ½ the deck thickness for Full-Depth Deck Repairs in areas of deck preparation (full-depth minus grinding if no deck preparation).

6.4.28 Removing Old Structure STA. XX + XX.XX

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

6.4.29 Anchor Assemblies for Steel Plate Beam Guard

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

6.4.30 Steel Diaphragms (Structure)

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

6.4.31 Welded Stud Shear Connectors X -Inch

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

6.4.32 Concrete Masonry Seal

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

6.4.33 Geotextile Fabric Type

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

6.4.34 Concrete Adhesive Anchors

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

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

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

6.4.36 Piling Steel Sheet Temporary

This quantity is used when the designer determines that retention of earth is necessary during excavation and soil forces require the design of steel sheet piling. This item is seldom used now that railroad excavations have a unique SPV.

Record this quantity to the nearest square foot for the area from the sheet pile tip elevation to one foot above the retained grade.

6.4.37 Temporary Shoring

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

Measured as square foot from the ground line in front of the shoring to a maximum of one foot above the retained grade. For the estimated quantity use the retained area (from the ground line in front of the shoring to the ground line behind the shoring, neglecting the additional height allowed for measurement).

6.4.38 Concrete Masonry Deck Repair

Record this quantity to the nearest cubic yard. Use 2-inch thickness for each Preparation area and ½ the deck thickness for Full-Depth Deck Repairs.

6.4.39 Sawing Pavement Deck Preparation Areas

Use 10 lineal feet per SY of Preparation Decks Type 1.

6.4.40 Removing Bearings

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

6.4.41 Ice Hot Weather Concreting

Used to provide a mechanism for payment of ice during hot weather concreting operations. See FDM 19-5-3.2 for bid item usage guidance and quantity calculation guidance. Bid as LB and round to the nearest 5 lbs.

6.4.42 Asphaltic Overlays

Estimate the overlay quantity by using the theoretical average overlay thickness and add ½" for variations in the deck surface. Provide this average thickness on the plan, as well. Use 110 lbs/(square yard - inch) to calculate hot mix asphalt (HMA) and polymer modified asphalt (PMA) overlay quantities.

For HMA overlays use 0.07 gallons/square yard to calculate tack coat quantity, unless directed otherwise.

Coordinate asphaltic quantity assumptions with the Region and roadway designers.

Table of Contents

9.1 General

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

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

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

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

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

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

9.2 Concrete

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

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

- Decks, Diaphragms, Overlays, Curbs, Parapets, Medians, Sidewalks and Concrete Slab Bridges (f'_c = 4 ksi)
- Other cast-in-place structures such as Culverts, Cantilever Retaining Walls and Substructure units (f'_c = 3.5 ksi)
- Other types of Retaining Walls (f'_c - values as specified in Chapter 14)
- Prestressed "I" Girders (f'_c = 6 to 8 ksi)
- Prestressed Box Girders (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 [C5.4.2.4]**. 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 concrete strength and concrete density, 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

Notice: Section 9.3 contents and the **LRFD [**article numbers**]** referenced in this Section are based on AASHTO LRFD Bridge Design Specifications (7th Edition – 2014).

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

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

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, use epoxy coated bars in the parapets and in the wing walls. For A3 abutments, use epoxy coated bars in the paving block and in the abutment backwall. For A1(fixed) abutments, use epoxy coated dowel bars.

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 deck pour and maintain the proper joint elevations.

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

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

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

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

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

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

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

9.3.1 Development Length and Lap Splices for Deformed Bars

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

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

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

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

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

The required development length, ℓ_{dh}, for bars in tension terminating in a standard hook is detailed in **LRFD [5.11.2.4]**. This length increases with the bar size. The basic development length, ℓ_{hb}, for a hooked bar is a function of bar diameter, d_b, and concrete strength, f'_c. The basic development length is multiplied by applicable modification factors to produce the required development length, ℓ_{dh}. Figure 9.9-2 shows typical development lengths for standard hooks in tension.

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

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

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

9.3.2 Bends and Hooks for Deformed Bars

Figure 9.9-1 shows standard hook and bend details for development of longitudinal tension reinforcement. Figure 9.9-3 shows standard hook and bend details for transverse reinforcement (open stirrups and ties). Figure 9.9-4 shows details for transverse reinforcement (closed stirrups). Dimensions for the bending details are shown as out to out of bar, as stated in the *Standard Specifications* Section 505.3.2. The diameter of a bend, measured on the inside of the bar for a standard bend is specified in **LRFD [5.10.2.3]**. Where a larger bend radius is required (non-standard bend) show the inside bend radius on the bar detail. When computing total bar lengths account for the accumulation in length in the bends. Use the figures mentioned above to account for accumulation in length for standard hooks and bends. One leg of bent bars is not dimensioned so that the tolerance for an error in the length due to bending is placed there. Fabrication tolerances for bent bars are specified in the *Concrete Reinforcing Steel Institute (CRSI) Manual of Standard Practices* or the *American Concrete Institute (ACI) Detailing Manual* as stated in Section 505.2.1 of the *Standard Specifications*.

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

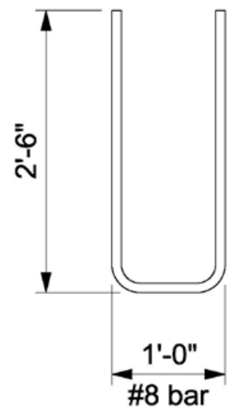


Figure 9.3-1
Bar Bend Detail (#8 bar)

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

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

9.3.3 Bill of Bars

Figure 9.9-5 shows a sample Bill of Bars table for a concrete slab. Different bar letter designations are used for abutments, slabs, and culverts, etc. If bundled bars are used, place a symbol adjacent to the bar mark of the bundled bars and a note below the Bill of Bars table stating the symbol represents bars to be bundled. A column for Bar Series is included if bars are cut.

9.3.4 Bar Series

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

The following general rules apply to the Bar Series table:

- Equal spacing of bars is required.
- There may be more than 1 Series with same number of bars.

- The total length of a bar is 60 feet (maximum).
- The minimum number of bars per Series is 4.
- Bent bars are bent after cutting.
- Set numbers are assigned to each Series used.

Figure 9.9-6 provides a sample layout for a Bar Series table. The Bill of Bars table will show the number of bars and the average bar length in the Series.

9.4 Steel

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

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

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

Structural carbon steel (ASTM A709 Grade 36) is used in components that are part of railings, and for steel diaphragms attached to prestressed girders. Structural carbon steel (ASTM A1011 Grade 36) is used in laminated elastomeric bearings. Structural carbon steel (ASTM A36) is used in components that are part of drains. The minimum yield strength is 36 ksi.

High strength structural steel (ASTM A709 Grade 50) is used in components that are part of railings and laminated elastomeric bearings, and (ASTM A572 Grade 50) is used in H-piles. High strength structural weathering steel (ASTM A709 Grade 50W) is used in bearings. The minimum yield strength is 50 ksi.

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

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

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

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

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

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

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

9.7 Miscellaneous Materials

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

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

Other materials or products used on highway structures are:

- Downspouts for Type GC and H drains may be fabricated from fiberglass conforming to ASTM D2996, Grade 1, Class A.
- Elastomeric bearing pads (non-laminated) are primarily used with prestressed "I" girders at fixed abutments and piers and at semi-expansion abutments. They are also used with prestressed "slab and box" sections at all supports. The requirements for these pads are described in Section 506.2.6 of the *Standard Specifications*.
- Elastomeric bearing pads (laminated) are primarily used with prestressed "I" girders at expansion supports. The requirements for these pads are described in Section 506.2.6 of the *Standard Specifications*.
- Preformed fillers are placed vertically in the joint between wing and diaphragm in A1 and A5 abutments, in the joint between wing and barrel in box culverts and at expansion joints in concrete cast-in-place retaining walls. Preformed fillers are placed along the front top surface of A1 and A5 abutments, along the outside top surfaces of fixed piers and under flanges between elastomeric bearing pad (non-laminated) and top edge of support. Cork filler is placed vertically on semi-expansion abutments. The requirements for fillers are described in Section 502.2.7 of the *Standard Specifications*.
- Polyethelene sheets are placed on the top surface of semi-expansion abutments to allow movement of the superstructure across the bearing surface. They are also placed between the structural approach slab and the subgrade, and the approach slab and its footing.

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

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

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

- Preformed Fabric, Class A, has been used as a bearing pad under steel bearings. The requirement for this material is given in Section 506.2.8.4 of the *Standard Specifications*.

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

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

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

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

9.8 Painting

All highway grade separation structures require steel girders to be painted because unpainted steel is subject to additional corrosion from vehicle salt spray. Additional discussion on painting is presented in Chapter 24 – Steel Girder Structures. The current paint system used for I-girders is the three-coat epoxy system specified in Section 517 of the *Standard Specifications*. Tub girders utilize a two-coat polysiloxane system, which includes painting of the inside of the tubs.

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

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

Table 9.8-1
Standard Colors for Steel Girders

[1] Shop applied color for weathering steel.

Federal Standard No. 595C can be found at www.federalstandardcolor.com/

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

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

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

The primary reason for painting steel structures is for the protection of the steel surface. Appearance is a secondary function that is maintained by using compatible top coatings over epoxy systems. Regarding appearance with respect to color retention, black is good, blues and greens are decent, and reddish browns are acceptable, but not the best. Reds are highly discouraged and should not be used.

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

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

Refer to Section 1.3.13 of the *Wisconsin Structure Inspection Manual* for the criteria covering spot painting versus complete painting of existing structures. This Section provides information for evaluating the condition of a paint system and recommended maintenance.

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

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

Table 9.8-2
Standard Colors for Concrete

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

9.9 Bar Tables and Figures

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

Table 9.9-1

Tension Lap Splice Length or Development Length - Deformed Bars

LRFD [5.11.2.1, 5.11.5.3.1] – 7th Edition (2014)

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

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

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

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

(f'_c = 4000 psi; f_y = 60 ksi)

BAR SPACING	BAR SIZE		4	5	6	7	8	9	10	11	TYPE
6” OR MORE	CLASS A 1.0 ℓ_d	TOP [1]	1-2	1-5	1-9	2-2	2-10	3-6	4-6	5-6	UNCOATED
			1-5	1-9	2-1	2-7	3-5	4-3	5-5	6-8	EPOXY
		OTHERS	1-0	1-0	1-3	1-6	2-0	2-6	3-3	3-11	UNCOATED
			1-0	1-6	1-10	2-3	3-0	3-9	4-10	5-11	EPOXY
	CLASS B 1.3 ℓ_d	TOP [1]	1-6	1-10	2-3	2-9	3-8	4-7	5-10	7-2	UNCOATED
			1-9	2-3	2-8	3-4	4-5	5-7	7-1	8-8	EPOXY
		OTHERS	1-1	1-4	1-7	2-0	2-7	3-3	4-2	5-1	UNCOATED
			1-3	2-0	2-5	3-0	3-11	4-11	6-3	7-8	EPOXY
	CLASS C 1.7 ℓ_d	TOP [1]	1-11	2-5	2-11	3-7	4-9	6-0	7-7	9-4	UNCOATED
			2-4	2-11	3-6	4-5	5-9	7-3	9-3	11-4	EPOXY
		OTHERS	1-5	1-9	2-1	2-7	3-5	4-3	5-5	6-8	UNCOATED
			1-8	2-7	3-1	3-10	5-1	6-5	8-2	10-0	EPOXY
LESS THAN 6”	CLASS A 1.0 ℓ_d	TOP [1]	1-5	1-9	2-2	2-8	3-6	4-5	5-7	6-10	UNCOATED
			1-9	2-2	2-7	3-3	4-3	5-4	6-9	8-4	EPOXY
		OTHERS	1-0	1-3	1-6	1-11	2-6	3-2	4-0	4-11	UNCOATED
			1-3	1-11	2-3	2-10	3-9	4-9	6-0	7-4	EPOXY
	CLASS B 1.3 ℓ_d	TOP [1]	1-10	2-4	2-9	3-5	4-6	5-9	7-3	8-11	UNCOATED
			2-3	2-10	3-4	4-2	5-6	6-11	8-10	10-10	EPOXY
		OTHERS	1-4	1-8	2-0	2-6	3-3	4-1	5-2	6-5	UNCOATED
			1-7	2-6	3-0	3-8	4-10	6-2	7-9	9-7	EPOXY
	CLASS C 1.7 ℓ_d	TOP [1]	2-5	3-0	3-7	4-6	5-11	7-6	9-6	11-8	UNCOATED
			2-11	3-8	4-5	5-6	7-2	9-1	11-6	14-2	EPOXY
		OTHERS	1-9	2-2	2-7	3-3	4-3	5-4	6-9	8-4	UNCOATED
			2-1	3-3	3-10	4-10	6-4	8-0	10-2	12-6	EPOXY

Table 9.9-2

Tension Lap Splice Length or Development Length – Deformed Bars

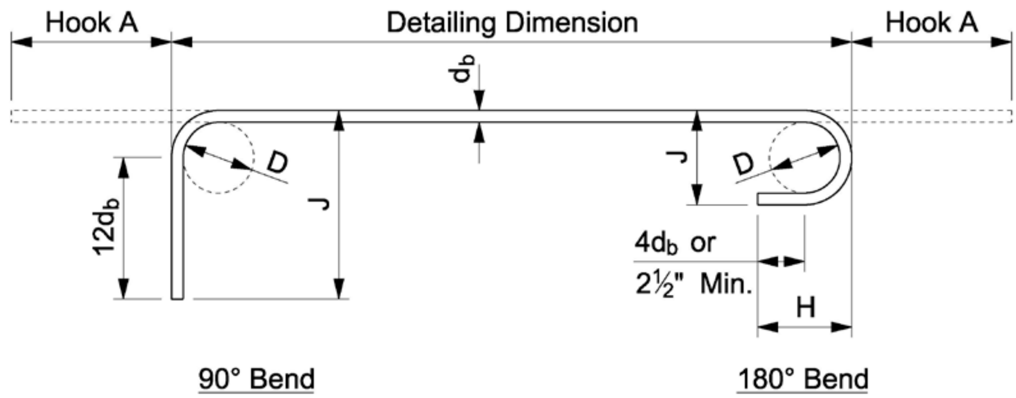
LRFD [5.11.2.1, 5.11.5.3.1] – 7th Edition (2014)

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

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

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

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



d_b = nominal diameter of reinforcing bar (in)

Definition of standard hooks **LRFD [5.10.2.1, C5.11.2.4.1] – 7th Edition (2014)**

MINIMUM BEND DIAMETER (D) – **LRFD [5.10.2.3] – 7th Edition (2014)**

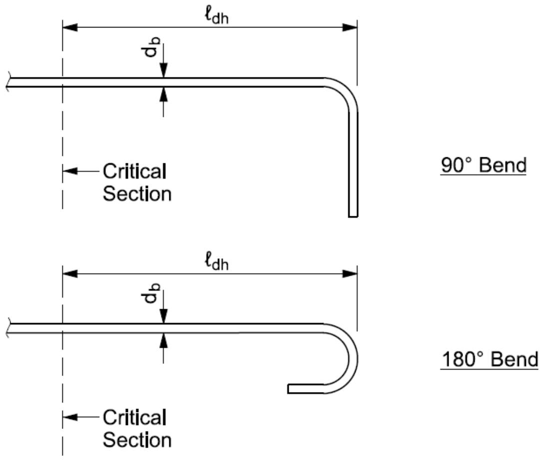
$D = 6d_b$ FOR #3 THRU #8

$D = 8d_b$ FOR #9, #10, and #11

BAR SIZE	MINIMUM HOOK, ALL GRADES					
	90° HOOKS			180° HOOKS		
	HOOK A	J	J MINUS HOOK A [1]	HOOK A	J	APPROX. H
4	7	8	1	6	4	4 ½
5	8 ½	10	1 ½	7	5	5
6	10	1-0	2	8	6	6
7	1-0	1-2	2	10	7	7
8	1-1 ½	1-4	2 ½	11	8	8
9	1-4	1-7	3	1-3	11 ¼	10 ¼
10	1-6	1-9 ½	3 ½	1-5	1-0 ¾	11 ½
11	1-8	2-0	4	1-7	1-2 ¼	1-0 ¾

Figure 9.9-1
Standard Hooks and Bends for Deformed Longitudinal Reinforcement

[1] "J" MINUS "HOOK A" = DEDUCTION FOR ONE BEND



f'c=3.5 ksi; fy=60 ksi		
Bar Size	ℓ_{dh}	
	Uncoated $\ell_{hb}(0.7)$	Epoxy $\ell_{hb}(0.7)(1.2)$
3	0' - 6"	0' - 7"
4	0' - 8"	0' - 9"
5	0' - 9"	0' - 11"
6	0' - 11"	1' - 1"
7	1' - 1"	1' - 3"
8	1' - 3"	1' - 6"
9	1' - 5"	1' - 8"
10	1' - 7"	1' - 10"
11	1' - 9"	2' - 1"

Figure 9.9-2
Development Length for Standard Hooks in Tension
(See Figure 9.9-1 for bend details)

The development length for standard hooks in tension, ℓ_{dh}, shall not be less than the product of the basic tension development length, ℓ_{hb}, and the appropriate modification factor(s), λ_i, $8d_b$, or 6-inches. The following equation is for the required development length for standard hooks in tension (in):

$\ell_{dh} = \max(\ell_{hb}\,\lambda_i,\ 8d_b,\ 6.0)$ **LRFD [5.11.2.4.1] – 7th Edition (2014)**

Where:

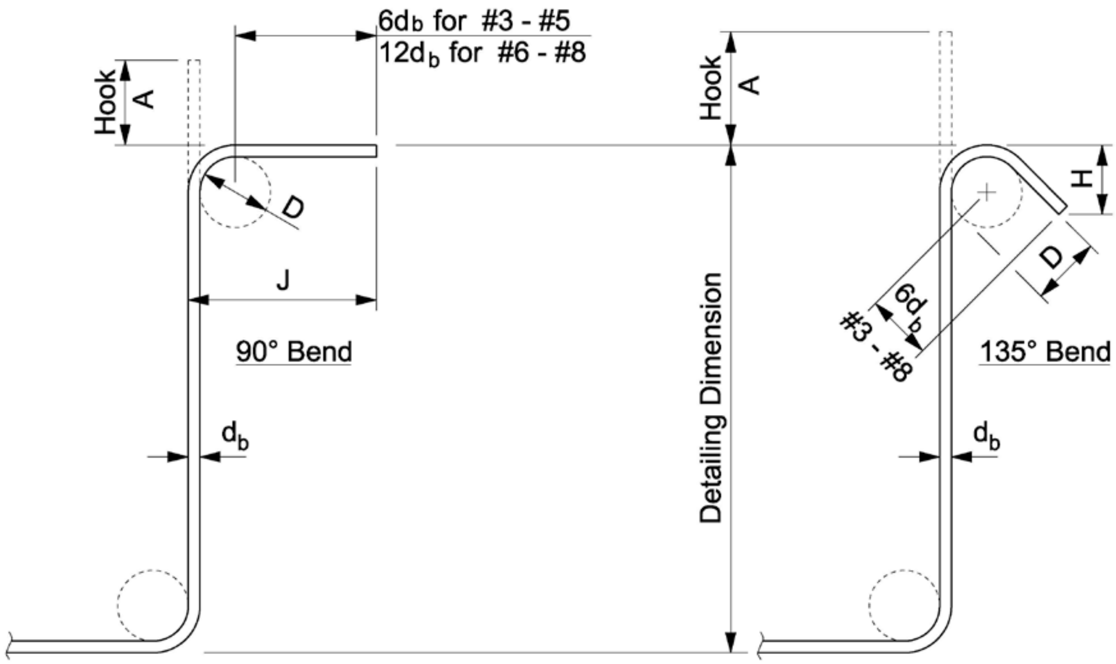
$\ell_{hb} = 38d_b / (f'_c)^{1/2}$ = basic hook development length (in.) **LRFD [Eq'n 5.11.2.4.1-1]**

λ_i = modification factor(s) **LRFD [5.11.2.4.2] – 7th Edition (2014)**

- (0.70) Side cover for #11 bar and smaller, normal to plane of hook, is not less than 2.5 inches, and 90 hook, cover on bar extension beyond hook not less than 2.0 inches
- (0.80) Hooks for #11 bar and smaller enclosed vertically or horizontally within ties or stirrups ties which are placed along the full development length, ℓ_{dh}, at a spacing not exceeding $3d_b$
- (1.20) Epoxy coated reinforcement

d_b = diameter of bar (in.)

f'_c = specified compressive strength of concrete (ksi)



d_b = nominal diameter of reinforcing bar (in)

Definition of Standard Hooks **LRFD [5.10.2.1] – 7th Edition (2014)**

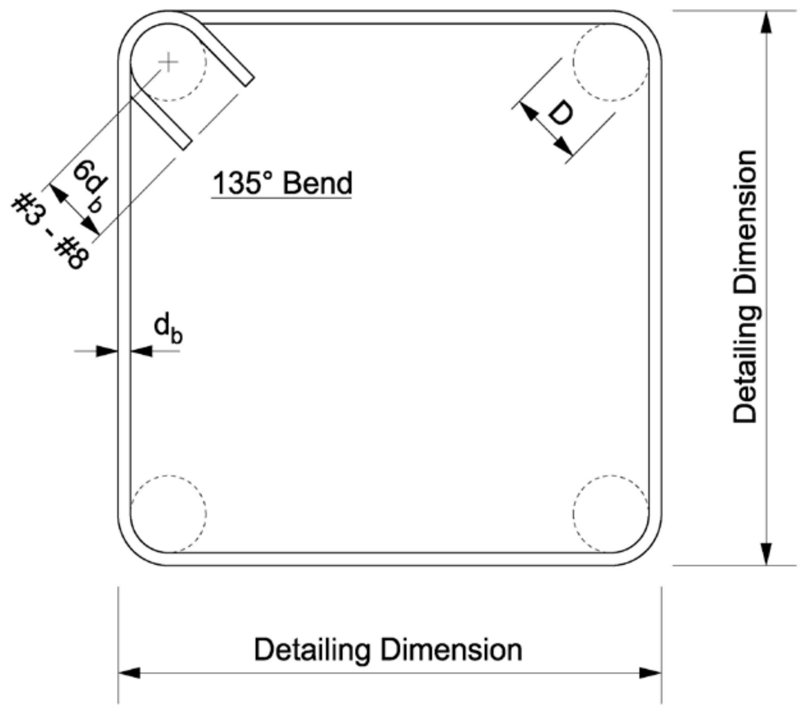
MINIMUM BEND DIAMETER (D) – **LRFD [5.10.2.3] – 7th Edition (2014)**

D = $4d_b$ FOR #3 THRU #5

D = $6d_b$ FOR #6 THRU #8

MINIMUM HOOK, ALL GRADES					
		90° HOOKS		135° HOOKS	
BAR SIZE	D	HOOK A	APPROX J	HOOK A	H
3	1 ½	3	4	4	2 ½
4	2	3 ½	4 ½	4 ½	3
5	2 ½	4 ½	6	5 ½	3 ¾
6	4 ½	10	1-0	8	4 ½

Figure 9.9-3
Standard Hooks and Bends for Deformed Transverse Reinforcement
(Stirrups and Ties)



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

d_b = nominal diameter of reinforcing bar (in)

Definition of Standard Hooks **LRFD [5.10.2.1] – 7th Edition (2014)**

MINIMUM BEND DIAMETER (D) – **LRFD [5.10.2.3] – 7th Edition (2014)**

D = 4d_b FOR #3 THRU #5

D = 6d_b FOR #6 THRU #8

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

Figure 9.9-4
Standard Details and Bends for Deformed Transverse Reinforcement
(Closed Stirrups)

9.12 Appendix - Draft Bar Tables

The following Draft Bar Tables are provided for information only. We expect the tables to be moved into the main text of Chapter 9 in January of 2019, and at that time to begin their use. We are delaying their use to allow time for modification of details and software that are affected.

The 2015 Interim Revisions to the AASHTO LRFD Bridge Design Specifications (7th Edition), modified the tension development lengths and tension lap lengths for straight deformed bars as follows - (**LRFD [**article number**]** references below match the AASHTO LRFD Bridge Design Specifications – 8th Edition):

The tension development length, ℓ_d ,shall not be less than the product of the basic tension development length, ℓ_{db} , and the appropriate modification factors, λ_i . **LRFD [5.10.8.2.1a]**

$$\ell_d = \ell_{db} \cdot (\lambda_{rl} \cdot \lambda_{cf} \cdot \lambda_{rc} \cdot \lambda_{er}) / \lambda$$

in which: $\ell_{db} = 2.4 \cdot d_b \cdot [f_y / (f'_c)^{1/2}]$

where:

ℓ_{db} = basic development length (in.)

λ_{rl} = reinforcement location factor

λ_{cf} = coating factor

λ = conc. density modification factor ; for normal weight conc. = 1.0 , **LRFD [5.4.2.8]**

λ_{rc} = reinforcement confinement factor

λ_{er} = excess reinforcement factor

f_y = specified minimum yield strength of reinforcement (ksi)

d_b = nominal diameter of reinforcing bar (in.)

f'_c = compressive strength of concrete for use in design (ksi)

Top bars will continue to refer to horizontal bars placed with more than 12” of fresh concrete cast below it. Bars not meeting this criteria will be referred to as Others.

Per **LRFD [5.10.8.4.3a]**, there are two lap splice classes, Class A and Class B.

- Class A lap splice ……………………1.0 ℓ_d
- Class B lap splice …………………... 1.3 ℓ_d

The criteria for where to apply each Class is covered in the above reference.

Draft Table

Epoxy Coated (f'_c = 4,000 psi; f_y = 60,000 psi)

Basic Lap - Others					
Bar Size	Min. Cover	Class A (1.0 ℓ_d) s < 6" cts.	Class A (1.0 ℓ_d) s ≥ 6" cts.	Class B (1.3 ℓ_d) s < 6" cts.	Class B (1.3 ℓ_d) s ≥ 6" cts.
4	1.5"	1'-6"	1'-6"	1'-11"	1'-11"
	2.0"	1'-6"	1'-6"	1'-11"	1'-11"
	≥ 2.5"	1'-6"	1'-6"	1'-11"	1'-11"
5	1.5"	2'-3"	2'-3"	3'-0"	3'-0"
	2.0"	2'-3"	1'-10"	3'-0"	2'-4"
	≥ 2.5"	2'-3"	1'-10"	3'-0"	2'-4"
6	1.5"	2'-11"	2'-9"	3'-10"	3'-7"
	2.0"	2'-11"	2'-9"	3'-10"	3'-7"
	≥ 2.5"	2'-11"	2'-2"	3'-10"	2'-10"
7	1.5"	3'-7"	3'-7"	4'-8"	4'-8"
	2.0"	3'-6"	3'-2"	4'-6"	4'-2"
	≥ 2.5"	3'-6"	3'-2"	4'-6"	4'-2"
8	1.5"	4'-6"	4'-6"	5'-11"	5'-11"
	2.0"	4'-6"	3'-8"	5'-11"	4'-9"
	≥ 2.5"	4'-6"	3'-8"	5'-11"	4'-9"
9	1.5"	5'-9"	5'-7"	7'-5"	7'-4"
	2.0"	5'-9"	4'-6"	7'-5"	5'-10"
	≥ 2.5"	5'-9"	4'-1"	7'-5"	5'-4"
10	1.5"	7'-4"	6'-9"	9'-7"	8'-10"
	2.0"	7'-4"	5'-6"	9'-7"	7'-2"
	≥ 2.5"	7'-4"	4'-10"	9'-7"	6'-3"
11	1.5"	9'-1"	8'-2"	11'-9"	10'-7"
	2.0"	9'-1"	6'-8"	11'-9"	8'-7"
	≥ 2.5"	9'-1"	6'-0"	11'-9"	7'-9"

Epoxy Coated (f'_c = 4,000 psi; f_y = 60,000 psi)

Horizontal Lap w/ >12" Concrete Cast Below - Top					
Bar Size	Min. Cover	Class A (1.0 ℓ_d) s < 6" cts.	Class A (1.0 ℓ_d) s ≥ 6" cts.	Class B (1.3 ℓ_d) s < 6" cts.	Class B (1.3 ℓ_d) s ≥ 6" cts.
4	1.5"	1'-11"	1'-11"	2'-6"	2'-6"
	2.0"	1'-11"	1'-11"	2'-6"	2'-6"
	≥ 2.5"	1'-11"	1'-11"	2'-6"	2'-6"
5	1.5"	2'-7"	2'-7"	3'-4"	3'-4"
	2.0"	2'-7"	2'-4"	3'-4"	3'-1"
	≥ 2.5"	2'-7"	2'-4"	3'-4"	3'-1"
6	1.5"	3'-4"	3'-1"	4'-4"	4'-0"
	2.0"	3'-4"	3'-1"	4'-4"	4'-0"
	≥ 2.5"	3'-4"	2'-10"	4'-4"	3'-8"
7	1.5"	4'-1"	4'-1"	5'-3"	5'-3"
	2.0"	4'-0"	3'-7"	5'-2"	4'-8"
	≥ 2.5"	4'-0"	3'-7"	5'-2"	4'-8
8	1.5"	5'-2"	5'-2"	6'-8"	6'-8"
	2.0"	5'-2"	4'-1"	6'-8"	5'-4"
	≥ 2.5"	5'-2"	4'-1"	6'-8"	5'-4"
9	1.5"	6'-6"	6'-4"	8'-5"	8'-3"
	2.0"	6'-6"	5'-1"	8'-5"	6'-7"
	≥ 2.5"	6'-6"	4'-8"	8'-5"	6'-0"
10	1.5"	8'-4"	7'-8"	10'-10"	10'-0"
	2.0"	8'-4"	6'-3"	10'-10"	8'-1"
	≥ 2.5"	8'-4"	5'-6"	10'-10"	7'-1"
11	1.5"	10'-3"	9'-3"	13'-4"	12'-0"
	2.0"	10'-3"	7'-6"	13'-4"	9'-9"
	≥ 2.5"	10'-3"	6'-10"	13'-4"	8'-10"

Table of Contents

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, geotechnical and 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 Geotechnical Investigation (see Chapter 10 – Geotechnical 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 sheet(s) 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 loads.
- Depth to suitable bearing material.
- Potential for liquefaction, undermining or scour.
- Frost potential.

activities pose a potential threat to nearby facilities. Contact the geotechnical engineer for further discussion and assistance, if vibrations appear to be a concern.

11.2 Shallow Foundations

11.2.1 General

Design of a shallow foundation, also known as a spread footing, must provide adequate resistance against geotechnical and structural failure. The design must also limit deformations to within tolerable values. This is true for designs using ASD or LRFD. In many cases, a shallow foundation is the most economical foundation type, provided suitable soil conditions exist within a depth of approximately 0 to 15 feet below the base of the proposed foundation.

WisDOT policy item:

Design shallow foundations in accordance with *AASHTO LRFD*. No additional guidance is available at this time.

Discussion is provided in 12.8 and 13.1 about design loads at abutments and piers, respectively. Live load surcharges at bridge abutments are described in 12.8.

11.2.2 Footing Design Considerations

The following design considerations apply to shallow foundations:

- Scour must not result in the loss of bearing or stability.
- Frost must not cause unacceptable movements.
- External or surcharge loads must be adequately supported.
- Deformation (settlement) and angular distortion must be within tolerable limits.
- Bearing resistance must be sufficient.
- Eccentricity requirements must be satisfied.
- Sliding resistance must be satisfied.
- Overall (global) stability must be satisfied.
- Uplift resistance must be sufficient.
- The effects of ground water must be mitigated and/or considered in the design.

11.2.2.1 Minimum Footing Depth

Foundation type selection and the preliminary design process require input from the geotechnical and hydraulic disciplines. The geotechnical engineer should provide guidance on the minimum embedment for shallow foundations that takes into consideration frost protection delaying critical structural connections such as closure pours or casting of decks that are continuous. Generally project timelines may restrict the time available for soil consolidation. Any project delays for geotechnical reasons must be thoroughly transmitted to, and analyzed by, design personnel.

11.2.2.3 Location of Ground Water Table

The location of the ground water table will impact both the stability and constructability of shallow foundations. A rise in the ground water table will cause a reduction in the effective vertical stress in soil below the footing and a subsequent reduction in the factored bearing resistance. A fluctuation in the ground water table is not usually a bearing concern at depths greater than 1.5 times the footing width below the bottom of footing.

WisDOT policy item:

The highest anticipated groundwater table should be used to determine the factored bearing resistance of footings. The Geotechnical Engineer should select this elevation based on the borings and knowledge of the specific site.

11.2.2.4 Sloping Ground Surface

The influence of a sloping ground surface must be considered for design of shallow foundations. The factored bearing resistance of the footing will be impacted when the horizontal distance is less than three times the footing width between the edge of sloping surface and edge of footing. Shallow foundations constructed in proximity to a sloping ground surface must be checked for overall stability. Procedures for incorporating sloping ground influence can be found in FHWA Publication SH-02-054, *Geotechnical Engineering Circular No. 6 Shallow Foundations* and **LRFD [10.6.3.1.2c]** Considerations for Footings on Slopes.

11.2.3 Settlement Analysis

Settlement should be computed using Service I Limit State loads. Transient loads may be omitted to compute time-dependent consolidation settlement. Two aspects of settlement are important to structural designers: total settlement and differential settlement (ie relative displacement between adjacent substructure units). In addition to the amount of settlement, the designer also needs to determine the time rate for it to occur.

Vertical settlement can be a combination of elastic, primary consolidation and secondary compression movement. In general, the settlement of footings on cohesionless soil, very stiff to hard cohesive soil and rock with tight, unfilled joints will be elastic and will occur as load is applied. For footings on very soft to stiff cohesive soil, the potential for primary consolidation and secondary compression settlement components should be evaluated in addition to elastic settlement.

The design of shallow foundations on cohesionless soil (sand, gravel and non-plastic silt), either as found in-situ or as engineered fill, is often controlled by settlement potential rather than bearing resistance, or strength, considerations. The method used to estimate settlement of footings on cohesionless soil should therefore be reliable so that the predicted settlement is rarely less than the observed settlement, yet still reasonably accurate so that designs are efficient.

Elastic settlement is estimated using elastic theory and a value of elastic modulus based on the results of in-situ or laboratory testing. Elastic deformation occurs quickly and is usually small. Elastic deformation is typically neglected for movement that occurs prior to placement of girders and final bridge connections.

Semi-empirical methods are the predominant techniques used to estimate settlement of shallow foundations on cohesionless soil. These methods have been correlated to large databases of simple and inexpensive tests such as the Standard Penetration Test (SPT) and the Cone Penetrometer Test (CPT).

Consolidation of clays or clayey deposits may result in substantial settlement when the structure is founded on cohesive soil. Settlement may be instantaneous or may take weeks to years to complete. Furthermore, because soil properties may vary beneath the foundation, the duration of the consolidation and the amount of settlement may also vary with the location of the footing, resulting in differential settlement between footing locations. The consolidation characteristics of a given soil are a function of its past history. The reader is directed to FHWA Publication SA-02-054, *Geotechnical Engineering Circular No. 6 Shallow Foundations* for a detailed discussion on consolidation theory and principles.

The rate of consolidation is usually of lesser concern for foundations, because superstructure damage will occur once the differential settlements become excessive. Shallow foundations are designed to withstand the settlement that will ultimately occur during the life of the structure, regardless of the time that it takes for the settlement to occur.

The design of footings bearing on intermediate geomaterials (IGM) or rock is generally controlled by considerations other than settlement. Intermediate geomaterial is defined as a material that is transitional between soil and rock in terms of strength and compressibility, such as residual soil, glacial till, or very weak rock. If a settlement estimate is necessary for shallow foundations supported on IGM or rock, a method based on elastic theory is generally the best approach. As with any of the methods for estimating settlement that use elastic theory, a major limitation is the engineer’s ability to accurately estimate the modulus parameter(s) required by the method.

11.2.4 Overall Stability

Overall stability of shallow foundations that are located on or near slopes is evaluated using a limiting equilibrium slope stability analysis. Both circular arc and sliding-block type failures are considered using a Modified Bishop, simplified Janbu, Spencers or simplified wedge analysis, as applicable. The Service I load combination is used to analyze overall stability. A free body diagram for overall stability is presented in Figure 11.2-1.

Detailed guidance to complete a limiting equilibrium analysis is presented in FHWA Publication NHI-00-045, *Soils and Foundation Workshop Reference Manual* and **LRFD [11.6.2.3]**.

11.3 Deep Foundations

When competent bearing soil is not present near the base of the proposed foundation, structure loads must be transferred to a deeper stratum by using deep foundations such as piles or drilled shafts (caissons). Deep foundations can be composed of piles, drilled shafts, micropiles or augered cast-in-place piles.

The primary functions of a deep foundation are:

- To transmit the load of the structure through a stratum of poor bearing capacity to one of adequate bearing capacity.
- To eliminate objectionable settlement.
- To transfer loads from a structure through erodible soil in a scour zone, to stable underlying strata.
- To anchor structures subjected to hydrostatic uplift or overturning forces.
- To resist lateral loads from earth pressures, as well as external forces.

11.3.1 Driven Piles

Deep foundation support systems have been in existence for many years. The first known pile foundations consisted of rows of timber stakes driven into the ground. Timber piles have been found in good condition after several centuries in a submerged environment. Several types of concrete piles were devised at the turn of the twentieth century. The earliest concrete piles were cast-in-place, followed by reinforced, precast and prestressed concrete piling. The requirement for longer piles with higher bearing capacity led to the use of concrete-filled steel pipe piles in about 1925. More recently, steel H-piles have also been specified due to ease of fabrication, higher bearing capacity, greater durability during driving and the ability to easily increase or decrease driven lengths.

11.3.1.1 Conditions Involving Short Pile Lengths

WisDOT policy generally requires piles to be driven a distance of 10 feet or greater below the original ground surface. Concern exists that short pile penetration in foundation materials of variable consistency may not adequately restrain lateral movements of substructure units. Pile penetrations of less than 10 feet are allowed if prebored at least 3 feet into solid rock. If conditions detailed in the Site Investigation Report clearly indicate that minimum pile penetration cannot be achieved, preboring should be included as a pay quantity. If there is a potential that preboring may not be necessary, do not include it in the plan documents. Piles which are not prebored into rock must not only meet the 10-foot minimum pile penetration criteria but must also have at least 5 feet of penetration through material with a blow count of at least 7 blows per foot. Piling should be “firmly seated” on rock after placement in prebored holes. The annular space between the cored holes in bedrock and piling should then be filled with concrete. Some sites may require casing during the preboring operation. If casing is required, it should be clearly indicated in the plan documents. Refer to 11.3.1.6 11.3.1.6 for additional information on preboring.

Foundations without piles (spread footings) should be given consideration at sites where pile penetrations of less than 10 feet are anticipated. The economics of the following two alternatives should be investigated:

1. Design for a shallow foundation founded at a depth where the foundation material is adequate. Embed the footing 6 inches into sound rock for lateral stability.

2. Excavate to an elevation where foundation material is adequate, and backfill to the bottom of footing elevation with suitable granular material or concrete.

If a substructure unit is located in a stream or lake, consideration should be given to the effects of the anticipated stream bed scour when selecting the footing type. Pile length computations should not incorporate pile resistance developed within the scour zone. The pile cross section should also be checked to ensure it can withstand the driving stresses necessary to penetrate through the anticipated scour depth and reach the required driving resistance plus the frictional resistance within the scour zone.

11.3.1.2 Pile Spacing

Arbitrary pile spacing rules specifying maximums and minimums are extensively used in foundation design. Proper spacing is dependent upon length, size, shape and surface texture of piles, as well as soil characteristics. A wide spacing of piles reduces heaving and possible uplifting of the pile, damage by tension due to heaving and the possibility of crushing from soil compression. Wider spacing more readily permits the tips of later-driven piles in the group to reach the same depths as the first piles and result in more even bearing and settlement. Large horizontal pressures are created when driving in relatively uncompressible strata, and damage may occur to piles already driven if piles are too closely spaced. In order to account for this, a minimum center-to-center spacing of 2.5 times the pile diameter is often required. **LRFD [10.7.1.2]** calls for a center-to-center pile spacing of not less than 2'-6" or 2.5 pile diameters (widths).

WisDOT policy item:

The minimum pile spacing is 2'-6" or 2.5 pile diameters, whichever is greater. For displacement piles located within cofferdams, or with estimated lengths ≥ 100 ft., the minimum pile spacing is 3.0 pile diameters. The minimum pile spacing for pile-encased piers and pile bents is 3'-0". The maximum pile spacing is 8'-0" for abutments, pile encased piers, and pile bents, based on standard substructure designs.

See Chapter 13 – Piers for criteria on battered piles in cofferdams. The distance from the side of any pile to the nearest edge of footing shall not be less than 9". Piles shall project at least 6" into the footings.

Depending on the type of concrete pile selected and the foundation conditions, the load-carrying capacity of the pile can be developed by shaft resistance, point resistance or a combination of both. Generally, driven concrete pile is employed as a displacement type pile.

When embedded in the earth, plain or reinforced concrete pile is generally not vulnerable to deterioration. The water table does not affect pile durability provided the concentration level is not excessive for acidity, alkalinity or chemical salt. Concrete pile that extends above the water surface is subject to abrasion damage from floating objects, ice debris and suspended solids. Deterioration can also result from frost action, particularly in the splash zone and from concrete spalling due to internal corrosion of the reinforcement steel. Generally, concrete spalls are a concern for reinforced concrete pile more than prestressed pile because of micro-cracks due to shrinkage, handling, placement and loading. Prestressing reduces crack width. Concrete durability increases with a corresponding reduction in aggregate porosity and water/cement ratio. WisDOT does not currently use prestressed reinforced concrete pile.

11.3.1.12.2.1 Driven Cast-In-Place Concrete Piles

Driven cast-in-place (CIP) concrete piles are formed by pouring concrete into a thin-walled closed-end steel shell which has been previously driven into the ground. A flat, oversize plate is typically welded to the bottom of the steel shell. Steel shells are driven either with or without a mandrel, depending on the wall thickness of the steel shell and the shell strength that is required to resist driving stress. Piling in Wisconsin is typically driven without the use of a mandrel. The minimum thickness of the steel shell should be that required for pile reinforcement and to resist driving stress. The Contractor may elect to furnish steel shells with greater thickness to permit their choice of driving equipment. A thin-walled shell must be carefully evaluated so that it does not collapse from soil pressure or deform from adjacent pile driving. Deformities or distortions in the pile shell could constrict the flow of concrete into the pile and produce voids or necking that reduce pile capacity. It is standard construction practice to inspect the open shell prior to concrete placement. Care must be exercised to avoid intermittent voids over the pile length during concrete placement.

Driven CIP concrete piles are considered a displacement-type pile, because the majority of the applied load is usually supported by shaft resistance. This pile type is frequently employed in slow flowing streams and areas requiring pile lengths of 50 to 120 feet. Driven CIP pile is generally selected over timber pile because of the availability of different diameters and wall thicknesses, the ability to adjust driven lengths and the ability to achieve greater resistances.

Driven CIP concrete piles may have a uniform cross section or may be tapered. The minimum cross-sectional area is required to be 100 and 50 square inches at the pile butt and tip, respectively. The Department has only used a limited number of tapered CIP piles and has experienced some driving problems with them.

For consistency with WisDOT design practice, the steel shell is ignored when computing the axial structural resistance of driven CIP concrete pile that is symmetrical about both principal axes. This nominal (ultimate) axial structural resistance capacity is computed using the following equation, neglecting the contribution of the steel shell to resist compression: **LRFD [Eq'n 5.6.4.4-3]**.

$$P_u \leq P_r = \phi P_n$$

Where:

$$P_n = 0.80(k_C \cdot f'_c \cdot (A_g - A_{st})) + f_y \cdot A_{st}$$

Where:

P_u	=	Factored axial force effect (kips)
P_r	=	Factored axial resistance without flexure (kips)
ϕ	=	Resistance factor
P_n	=	Nominal axial resistance without flexure (kips)
A_g	=	Gross area of concrete pile section ($inches^2$)
A_{st}	=	Total area of longitudinal reinforcement ($inches^2$)
k_c	=	Ratio of max. concrete compressive stress to specified compressive strength of concrete; $k_C = 0.85$ (for $f'_c \leq 10.0$ ksi)
f_y	=	Specified yield strength of reinforcement (ksi)
f'_c	=	Concrete compressive strength (ksi)

For cast-in-place concrete piles with steel shell and no steel reinforcement bars, A_{st} equals zero and the above equation reduces to the following.

$$P_n = 0.68 f'_c \, A_g$$

A resistance factor, ϕ, of 0.75 is used to compute the factored structural axial resistance capacity, as specified in **LRFD [5.5.4.2]**. For CIP piling there are no reinforcing ties, however the steel shell acts to confine concrete similar to ties.

$$P_r = 0.51 f_c \, A_g$$

For piles subject to large lateral loads, the structural pile capacity must also be checked for shear and combined stress against flexure and compression.

Piles subject to uplift must also be checked for tension resistance.

A concrete compressive strength of 4 ksi is the minimum value required by specification, while a value of 3.5 ksi is used in the structural design computations. Pile capacities are maximums, based on an assumed concrete compressive strength of 3.5 ksi. The concrete compressive strength of 3.5 ksi is based on construction difficulties and unknowns of placement. The

<table>
<tr><th colspan="3">Condition/Resistance Determination Method</th><th>Resistance Factor</th></tr>
<tr><td rowspan="7">Static Analysis – Used in Design Phase</td><td rowspan="3">Nominal Resistance of Single Pile in Axial Compression, φ_{stat}</td><td>Skin Friction and End Bearing in Clay and Mixed Soil
Alpha Method</td><td>0.35</td></tr>
<tr><td>Skin Friction and End Bearing in Sand
Nordlund/Thurman Method</td><td>0.45</td></tr>
<tr><td>Point Bearing in Rock</td><td>0.45</td></tr>
<tr><td>Block Failure, φ_{bl}</td><td>Clay</td><td>0.60</td></tr>
<tr><td rowspan="2">Uplift Resistance of Single Pile, φ_{up}</td><td>Clay and Mixed Soil
Alpha Method</td><td>0.25</td></tr>
<tr><td>Sand
Nordlund Method</td><td>0.35</td></tr>
<tr><td>Horizontal Resistance of Single Pile or Pile Group</td><td>All Soil Types and Rock</td><td>1.0</td></tr>
<tr><td colspan="2" rowspan="4">Nominal Resistance of Single Pile in Axial Compression – Dynamic Analysis – for the Hammer and Pile Driving System Actually - used During Construction for Pile Installation, φ_{dyn}</td><td>FHWA-modified Gates dynamic pile formula (end of drive condition only)</td><td>0.50 (1)</td></tr>
<tr><td>Wave equation analysis, without pile dynamic measurements or load test, at end of drive condition only</td><td>0.50</td></tr>
<tr><td>Driving criteria established by dynamic test [Pile Driving Analyzer, (PDA)] with signal matching [CAse Pile Wave Analysis Program, (CAPWAP)] at beginning of redrive conditions only, of at least two production pile per substructure, but no less than 2% of the structure production piles. Quality control of remaining piles by calibrated wave equation and/or dynamic testing.</td><td>0.65</td></tr>
<tr><td>Static Pile Load Test(s) and dynamic test (PDA) with signal matching (CAPWAP) at beginning of redrive conditions only, of at least two production pile per substructure, but no less than 2% of the structure production piles. Quality control of remaining piles by calibrated wave equation and/or dynamic testing.</td><td>0.80</td></tr>
</table>

(1) Based on department research and past experience

Table 11.3-1

Geotechnical Resistance Factors for Driven Pile

Resistance factors for structural design of piles are based on the material used, and are presented in the following sections of *AASHTO LRFD*:

- Concrete piles – **LRFD [5.5.4.2]**

- Steel piles – **LRFD [6.5.4.2]**

11.3.1.15 Bearing Resistance

A pile foundation transfers load into the underlying strata by either shaft resistance, point resistance or a combination of both. Any driven pile will develop some amount of both shaft and point resistance. However, a pile that receives the majority of its support capacity by friction or adhesion from the soil along its shaft is referred to as a friction pile, whereas a pile that receives the majority of its support from the resistance of the soil near its tip is a point resistance (end bearing) pile.

The design pile capacity is the maximum load the pile can support without exceeding the allowable movement criteria. When considering design capacity, one of two items may govern the design – the nominal (ultimate) geotechnical resistance capacity or the structural resistance capacity of the pile section. This section focuses primarily on the geotechnical resistance capacity of a pile.

The factored load that is applied to a single pile is carried jointly by the soil beneath the tip of the pile and by the soil around the shaft. The total factored load is not permitted to exceed the factored resistance of the pile foundation for each limit state in accordance with **LRFD [1.3.2.1 and 10.7.3.8.6]**. The factored bearing resistance, or pile capacity, of a pile is computed as follows:

$$\sum \eta_i \gamma_i Q_i \;\leq\; R_r \;=\; \varphi R_n \;=\; \varphi_{stat} R_p + \varphi_{stat} R_s$$

Where:

η_i	=	Load modifier
γ_i	=	Load factor
Q_i	=	Force effect (tons)
R_r	=	Factored bearing resistance of pile (tons)
R_n	=	Nominal resistance (tons)
R_p	=	Nominal point resistance of pile (tons)
R_s	=	Nominal shaft resistance of pile (tons)
φ	=	Resistance factor
φ_{stat}	=	Resistance factor for driven pile, static analysis method

This equation is illustrated in Figure 11.3-1.

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 [5.6.4.4]**

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:

						Modified Gates Driving Criteria		PDA/CAPWAP Driving Criteria	
Pile Size	Shell Thickness (inches)	Concrete or Steel Area (A_g or A_s) (in^2)	Nominal Resistance (Pn) (tons) (2)(3)(6)	(ϕ)	Maximum Factored Resistance (Pr) (tons) (4)	Factored Resistance (Pr) (ϕ = 0.50) (tons)	Required Driving Resistance (Rn_{dyn}) (tons) (5)	Factored Resistance (Pr) (ϕ = 0.65) (tons)	Required Driving Resistance (Rn_{dyn}) (tons) (5)
Cast in Place Piles									
10 ¾"	0.219	83.5	99.4	0.75	75	55 (8)	110	72 (8)	110
10 ¾"	0.250	82.5	98.2	0.75	74	65 (8)	130	75 (9)	115
10 ¾"	0.365	78.9	93.8	0.75	70	75 (9)	150	75 (9)	115
10 ¾"	0.500	74.7	88.8	0.75	67	75 (9)	150	75 (9)	115
12 ¾"	0.250	118.0	140.4	0.75	105	80 (8)	160	104 (8)	160
12 ¾"	0.375	113.1	134.6	0.75	101	105 (9)	210	104 (9)	160
12 ¾"	0.500	108.4	129.0	0.75	97	105 (9)	210	104 (9)	160
14"	0.250	143.1	170.3	0.75	128	85 (8)	170	111 (8)	170
14"	0.375	137.9	164.1	0.75	123	120 (8)	240	120	185
14"	0.500	132.7	158.0	0.75	118	120 (9)	240	120 (9)	185
16"	0.375	182.6	217.3	0.75	163	145 (8)	290	159	245
16"	0.500	176.7	210.3	0.75	158	160 (9)	320	159 (9)	245
H-Piles									
10 x 42	NA (1)	12.4	310.0	0.50	155	90	180 (10)	117	180 (10)
12 x 53	NA (1)	15.5	387.5	0.50	194	110	220 (10)	143	220 (10)
14 x 73	NA (1)	21.4	535.0	0.50	268	125	250 (10)	162	250 (10)

Table 11.3-5
Typical Pile Axial Compression Resistance Values

Notes:

1. NA – not applicable

2. For CIP Piles: Pn = 0.8 (k_C * f'c * Ag + fy * As) **LRFD [Eq'n 5.6.4.4-3]**. k_C = 0.85 (for $f'_C \leq$ 10.0 ksi). 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} * Fe * As) **LRFD [Eq'n 6.9.5.1-1]** (λ = 0 for piles embedded in the ground below the substructure, i.e. no unsupported lengths)

Fe = fy = yield strength of steel = 50,000 psi

4. Pr = ϕ * Pn

   ϕ = 0.75 (**LRFD [5.5.4.2]** 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} = 0.50 for construction driving criteria using modified Gates

     φ_{dyn} = 0.65 for construction driving criteria using PDA/CAPWAP

   - 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 (see note 10).

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. Rounded values utilize some pile shell capacity)

10. Rn_{dyn} 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 Rn_{dyn} 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.

Abutment
Abutment Example: 12 x 53 H-piles to an estimated length of 100 feet at a unit cost of $40/foot.
Modified Gates: RDR = 220 tons, FACR = 110 tons, Total Load on Abut = 980 tons, Number of Piles = 980 tons / 110 tons = 9 piles Total Cost = 9 piles x 100 feet x $40/ft = $36,000
PDA/CAPWAP: RDR = 220 tons, FACR = 143 tons, Load on Abut = 980 tons, Number of Piles = 980 tons / 143 tons = 7 piles, however because of maximum spacing requirements the design will need 8 piles. Pile Cost = 8 piles x 100 feet x $40/ft = $32,000 PDA Testing Cost = 2 piles/sub. x $700/pile = $1,400 PDA Restrike Cost = 2 piles/sub. x $600/pile = $1,200 CAPWAP Evaluation = 1 eval./sub. x $400/eval. = $400 Total Cost = $35,000
PDA/CAPWAP Cost = $1000/abutment **Note:** For a three span bridge, with 12 x 53 H-piles to an estimated length of 100 feet at a unit cost of $40/foot, PDA/CAPWAP would provide an estimated structure savings of $52,000. For a two span bridge, with 12 x 53 H-piles to an estimated length of 40 feet at a unit cost of $40/foot, PDA/CAPWAP would provide an estimated structure savings of $5,400. Bid prices based on 2014-2015 cost data.

Table 11.3-6
Economical Evaluation for Deep Foundations with Two Construction Monitoring Methods

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 **LRFD [10.8]** Drilled Shafts and *Drilled Shafts: Construction Procedures and LRFD Design Methods*. FHWA Publication NHI-10-016, FHWA GEC 010. 2010.

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 [5.6 and 5.7]**. 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 11.3-7 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.

<table>
<tr><th colspan="4">Condition/Resistance Determination Method</th><th>Resistance Factor</th></tr>
<tr><td rowspan="15">Static Analysis – Used in Design Phase</td><td rowspan="9">Nominal Resistance of Single-Drilled Shaft in Axial Compression, φ_{stat}</td><td>Shaft Resistance in Clay</td><td>Alpha Method</td><td>0.45</td></tr>
<tr><td>Point Resistance in Clay</td><td>Total Stress</td><td>0.40</td></tr>
<tr><td>Shaft Resistance in Sand</td><td>Beta Method</td><td>0.55</td></tr>
<tr><td>Point Resistance in Sand</td><td>O’Neill and Reese</td><td>0.50</td></tr>
<tr><td>Shaft Resistance in IGMs</td><td>O’Neill and Reese</td><td>0.60</td></tr>
<tr><td>Point Resistance in IGMs</td><td>O’Neill and Reese</td><td>0.55</td></tr>
<tr><td rowspan="2">Shaft Resistance in Rock</td><td>Horvath and Kenney
O’Neill and Reese</td><td>0.55</td></tr>
<tr><td>Carter and Kulhawy</td><td>0.50</td></tr>
<tr><td>Point Resistance in Rock</td><td>Canadian Geotech. Soc.
Pressuremeter Method
O’Neill and Reese</td><td>0.50</td></tr>
<tr><td>Block Failure, φ_{bl}</td><td colspan="2">Clay</td><td>0.55</td></tr>
<tr><td rowspan="3">Uplift Resistance of Single-Drilled Shaft, φ_{up}</td><td>Clay</td><td>Alpha Method</td><td>0.35</td></tr>
<tr><td>Sand</td><td>Beta Method</td><td>0.45</td></tr>
<tr><td>Rock</td><td>Horvath and Kenney
Carter and Kulhawy</td><td>0.40</td></tr>
<tr><td>Group Uplift Resistance, φ_{ug}</td><td colspan="2">Sand and Clay</td><td>0.45</td></tr>
<tr><td>Horizontal Geotechnical Resistance of Single Shaft or Pile Group</td><td colspan="2">All Soil Types and Rock</td><td>1.0</td></tr>
</table>

Table 11.3-7

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

For drilled shafts, the base geotechnical resistance factors in Table 11.3-7 assume groups containing two to four shafts, which are slightly redundant. For groups containing at least five elements, the base geotechnical resistance factors in Table 11.3-7 should be increased by 20%.

WisDOT policy item:

When a bent contains at least 5 columns (where each column is supported on a single drilled shaft) the resistance factors in Table 11.3-7 should be increased up to 20 percent for the Strength Limit State.

For piers supported on a single drilled shaft, the resistance factors in Table 11.3-7 should be decreased by 20 percent for the Strength Limit State. Use of single drilled shaft piers requires approval from the Bureau of Structures.

Resistance factors for structural design of drilled shafts are obtained from **LRFD [5.5.4.2]**.

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. 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]**

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12.3 Types of Abutment Support

Piles, drilled shafts and spread footings are the general types of abutment support used. This section provides a brief description of each type of abutment support.

WisDOT policy item:

Geotechnical and structural design of abutment supports shall be in accordance *AASHTO LRFD*. No additional guidance is available at this time.

12.3.1 Piles or Drilled Shafts

Most abutments are supported on piles to prevent abutment settlement. Bridge approach embankments are usually constructed of fill material that can experience settlement over several years. This settlement may be the result of the type of embankment material or the original foundation material under the embankment. By driving piles through the embankment and into the original ground, abutments usually do not settle with the embankment. A settling embankment may be resisted by the abutment piles through friction between the piles and fill material. The added load to friction piles and the need for preboring should be considered.

It is generally not necessary to prebore non-displacement piles for any fill depths, and it is not necessary to prebore displacement piles for fill depths less than 15 feet below the bottom of footing. However, for some problem soils this may not apply. See the soils report to determine if preboring is required. If required, the Special Provisions must be written with preboring guidelines.

Battered piles may cause more of a problem than vertical piles and are given special consideration. When driving battered piles, reduced hammer efficiency may be experienced, and battered piles should not be considered when negative skin friction loads are anticipated.

Fill embankments frequently shift laterally, as well as vertically. A complete foundation site investigation and information on fill material is a prerequisite for successful pile design.

Piles placed in prebored holes cored into rock do not require driving. The full design loading can be used if the hole is of adequate size to prevent pile hangups and to allow filling with concrete.

Piles in abutments are subject to lateral loads. The lateral resistance on a pile is usually determined from an acceptable level of lateral displacement and not the ultimate load that causes a stress failure in the pile. The lateral resistance on a pile may be more dependent on the material into which the pile is driven than on the pile type. See Chapter 11 – Foundation Support for a more thorough discussion of piles and allowable pile loads.

12.3.2 Spread Footings

Abutments on spread footings are generally used only in cut sections where the original soil can sustain reasonable pressures without excessive settlement. The bearing resistance is determined by the Geotechnical Section or the geotechnical consultant.

With improved procedures and better control of embankment construction, spread footings can be used successfully on fill material. It is important that construction be timed to permit the foundation material to consolidate before the spread footings are constructed. An advantage of spread footings is that the differential settlement between approach fills and abutments is minimized.

The use of spread footings is given greater consideration for simple-span bridges than for continuous-span bridges. However, under special conditions, continuous-span bridges can be designed for small amounts of settlement. Drainage for abutments on spread footings can be very critical. For these reasons, pile footings are usually preferred.

Lateral forces on abutments are resisted by passive earth pressure and friction between the soil and concrete. A shear key provides additional area on which passive earth pressure can act. A berm in front of the abutment may be necessary to prevent a shear failure in the soil along the slope.

12.6 Abutment Drainage and Backfill

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

12.6.1 Abutment Drainage

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

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

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

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

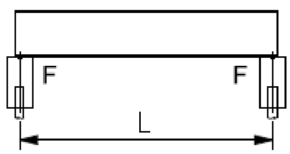
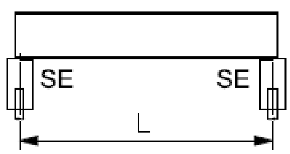
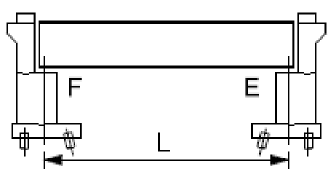
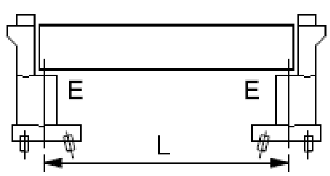
Pipe underdrains and weepholes may discharge water during freezing temperatures. In urban areas, this may create a problem due to the accumulation of flow and ice on sidewalks.

12.6.2 Abutment Backfill Material

All abutments and wings shall utilize “Backfill Structure” to facilitate drainage. See Standard Detail 9.01 – Structure Backfill Limits and Notes – for typical pay limits and plan notes.

12.7 Selection of Standard Abutment Types

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

Abutment Arrangements	Superstructures		
	Concrete Slab Spans	Prestressed Girders	Steel Girders
Type A1 (F-F) 	a. L ≤ 150' S ≤ 30° AL ≤ 50'	a. L ≤ 150' S ≤ 15° AL ≤ 50' 28" and 36" only (36W" thru 82W" require SE)	a. L ≤ 150' S ≤ 15° AL ≤ 50'
Type A1 (SE-SE) 	a. L ≤ 300' S ≤ 30° AL > 50'	a. L ≤ 300' S ≤ 40°	a. L ≤ 150' S ≤ 40°
Type A3 (F-E) 	Not used	Single span and (S > 40°)	Single span and (L > 150' or S > 40°)
Type A3 (E-E) 	b. L >300' and S ≤ 30° with rigid piers	L>300' or (S > 40° and multi-span)	Multi-span and (L>150' or S > 40°) with rigid piers

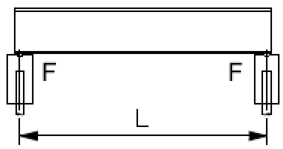
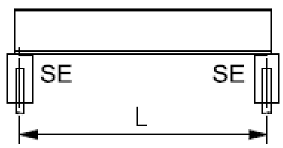
Abutment Arrangements	Superstructures		
	Concrete Slab Spans	Prestressed Girders	Steel Girders
Type A5 (F-F)  	L $\leq$ 150' S $\leq$ 30° AL $\leq$ 50'	L $\leq$ 150' S $\leq$ 15° AL $\leq$ 50'	L $\leq$ 150' S $\leq$ 15° AL $\leq$ 50'
Type A5 (SE-SE)  	L $\leq$ 200' S $\leq$ 30° AL > 50'	L $\leq$ 200' S $\leq$ 30°	L $\leq$ 150' S $\leq$ 30°
ABUTMENT TYPES			

Figure 12.7-1
Recommended Guide for Abutment Type Selection

Where:

S = Skew

AL = Abutment Length

F = Fixed seat

SE = Semi-Expansion seat

E = Expansion seat

L = Length of continuous superstructure between abutments

Footnotes to Figure 12.7-1:

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

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

12.8 Abutment Design Loads and Other Parameters

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

12.8.1 Application of Abutment Design Loads

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

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

Approach loads are not considered in the example below. However, designers shall include vertical reactions from reinforced concrete approaches as they directly transmit load from the approaches to the abutment. Reinforced concrete approaches include the concrete approach slab system (refer to FDM 14-10-15) and the structural approach slab system (as described in this chapter).

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

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

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

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

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

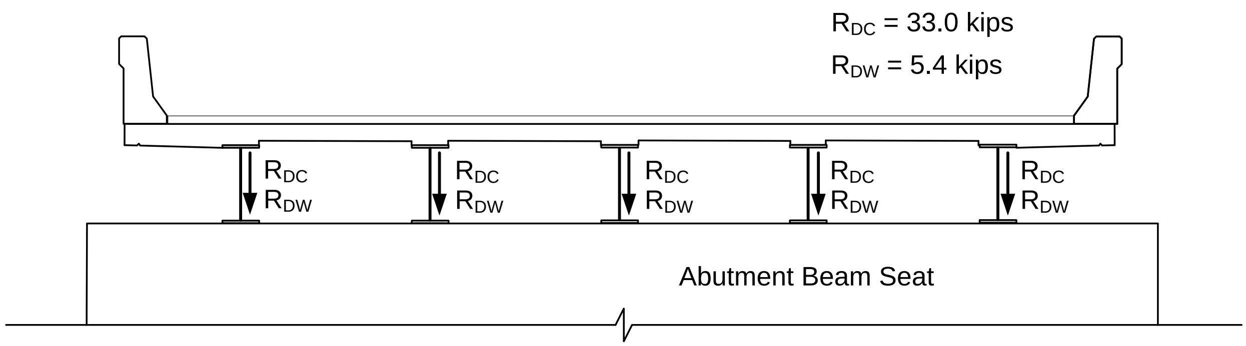


Figure 12.8-1
Dead Load on Abutment Beam Seat

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

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

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

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

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

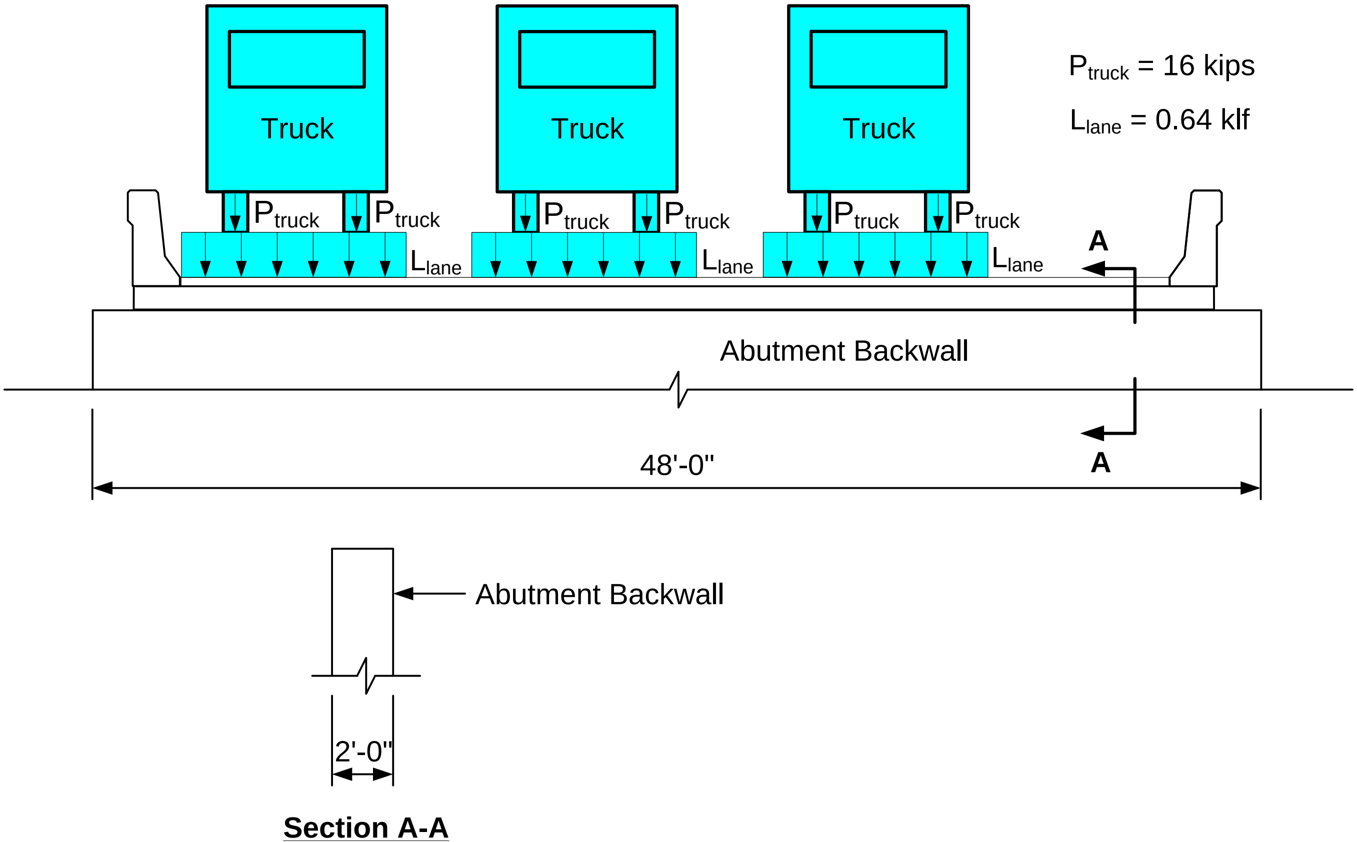


Figure 12.8-2
Live Load on Abutment Backwall

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

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

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

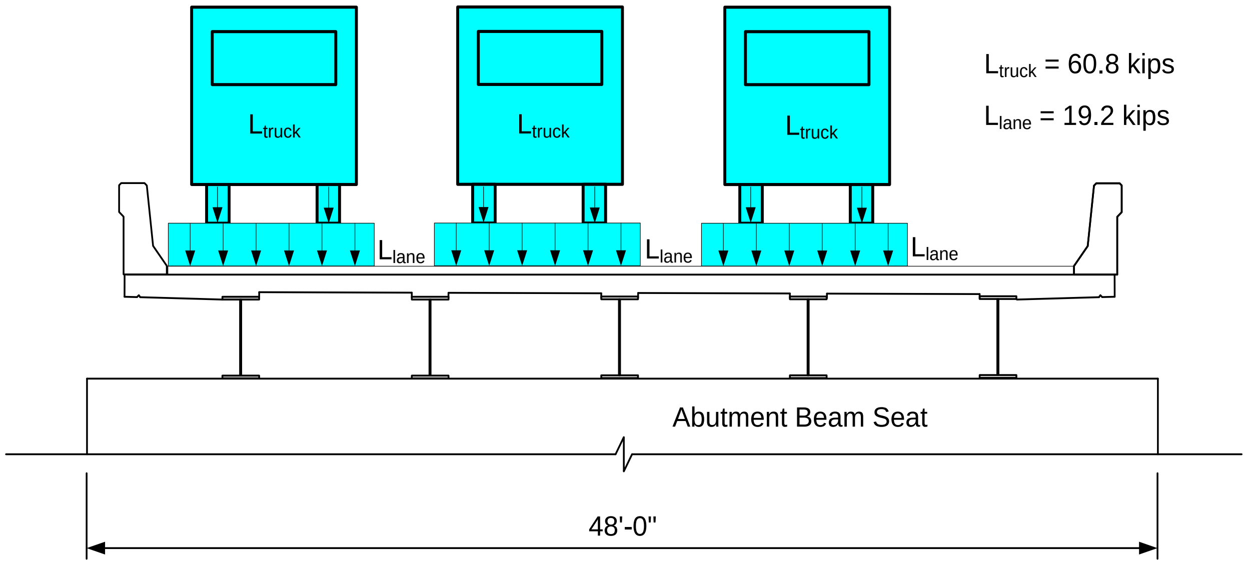


Figure 12.8-3
Live Load on Abutment Beam Seat

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

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

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

12.8.2 Load Modifiers and Load Factors

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

Description	Load Modifier
Ductility	1.00
Redundancy	1.00
Operational classification	1.00

Table 12.8-1
Load Modifiers Used in Abutment Design

Table 12.8-2 presents load factors used for abutment and wing wall design. Load factors presented in this table are based on the Strength I and Service I limit states. The load factors for WS and WL equal 0.00 for Strength I. Load factors for the Service I limit state for WS and WL are shown in the table below. Only apply these loads in the longitudinal direction.

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

Table 12.8-2
Load Factors Used in Abutment Design

* Use the minimum load factor for temperature and shrinkage unless checking for deformations.

12.8.3 Live Load Surcharge

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

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

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

WisDOT policy item:

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

For abutments without reinforced concrete approaches, the equivalent height of soil for vehicular loading on abutments shall be based on Table 12.8-3. For abutments with reinforced concrete approaches, one half of the equivalent height of soil shall be used to calculate the horizontal load on the abutment.

12.8.4 Other Abutment Design Parameters

The equivalent fluid unit weights of soils are as presented in **LRFD [Table 3.11.5.5-1]**. Values are presented for loose sand or gravel, medium dense sand or gravel, and dense sand or gravel. Values are also presented for level or sloped backfill and for at-rest or active soil conditions.

Table 12.8-4 presents other parameters used in the design of abutments and wing walls. Standard details are based on the values presented in Table 12.8-4.

Description	Value
Bottom reinforcing steel cover	3.0 inches
Top reinforcing steel cover	2.0 inches
Unit weight of concrete	150 pcf
Concrete strength, f'_c	3.5 ksi
Reinforcing steel yield strength, f_y	60 ksi
Reinforcing steel modulus of elasticity, E_s	29,000 ksi
Unit weight of soil	120 pcf
Unit weight of structural backfill	120 pcf
Soil friction angle	30 degrees

Table 12.8-4
Other Parameters Used in Abutment Design

12.8.5 Abutment and Wing Wall Design in Wisconsin

The standard details for abutments and wing walls were developed as an envelope of the loading conditions produced by the standard superstructure types, span lengths and geometric conditions presented in this manual. Prior BOS approval is required and special consideration should be given to designs that are outside of the limits presented in the standard details. The loading conditions, material properties and design methods presented in this chapter should be used for these special designs.

WisDOT policy items:

The resistance of the wing pile to horizontal forces should not be included in the calculations for the wing capacity.

The passive earth resistance can only be developed if there is significant movement of the wing. The soil under the wing may settle or otherwise erode. Therefore, the resistance of the soil friction and the passive earth pressure should not be utilized in resisting the forces on wing walls.

In computing the weight of the approach slab, assume there is settlement under the approach slab and place one-half of the weight of the slab on the abutment. An unfactored dead load value of 1.2 klf shall be used for concrete approach slabs and 2.0 klf for structural approach slabs. An unfactored live load value of 0.900 klf shall be applied to abutment approach slabs when used. Approach reactions shall act along the centroid of the foundation.

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

12.8.6 Horizontal Pile Resistance

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

Given information:

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

Back row pile spacing =	8.0 feet
Front row pile spacing =	5.75 feet
Ultimate Vertical Resistance, 12 3/4" CIP, Pr =	210 kips per pile
Factored Vertical Load on Front Row Pile*	160 kips per pile
Ultimate Horizontal Resistance of back row pile (from Geotech Report), Hr =	14 kips per pile
Ultimate Horizontal Resistance of front row pile (from Geotech Report), Hr =	11 kips per pile

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

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

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

$Hr_{battered}$ = 38.8 kips per pile

Calculate ultimate resistance provided by the pile configuration:

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

Hr = 10.4 klf

Hr > Hu = 10.3 klf OK

12.9 Abutment Body Details

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

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

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

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

12.9.1 Construction Joints

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

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

WisDOT exception to AASHTO:

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

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

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

12.9.2 Beam Seats

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

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

See the abutment standards for additional reinforcing required when beam seats are 4” higher, or more, than the lowest beam seat.

12.10 Timber Abutments

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

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

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

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

12.11 Bridge Approach Design and Construction Practices

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

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

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

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

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

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

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

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

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

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

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

- Use of select materials

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

- Strict control of moisture and density

- Proper compaction

- Installation of moisture barriers

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

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

The geotechnical engineer should evaluate approaches for settlement susceptibility and provide recommendations for mitigating settlements prior to approach placement. The bridge designer should determine if a structural approach slab is required and coordinate details with the roadway engineer. Usage of structural approach slabs is currently based on road functional classifications and considerations to traffic volumes (AADT), design speeds, and settlement susceptibility. Structural approach slabs are not intended to mitigate excessive approach settlements.

WisDOT policy item:

Structural approach slabs shall be used on all Interstate and US highway bridges. Structural approach slabs are recommended for bridges carrying traffic volumes greater than 3500 AADT in the future design year. Structural approach slabs are not required on buried structures and culverts. Structural approach slabs should not be used on rehabilitation projects, unless approved otherwise. Other locations can be considered with the approval of the Chief Structural Design Engineer. Design exceptions to structural approach slabs are considered on a project-by-project basis.

Standards for Structural Approach Slab for Type A1 and A3 Abutments and Structural Approach Slab Details for Type A1 and A3 Abutments are available for guidance.

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13.7 Hammerhead Pier Cap Design

WisDOT policy item:

Hammerhead pier caps shall be designed using the strut-and-tie method **LRFD [5.8.2]**.

The strut-and-tie method (STM) is simply the creation of an internal truss system used to transfer the loads from the bearings through the pier cap to the column(s). This is accomplished through a series of concrete "struts" that resist compressive forces and steel "ties" that resist tensile forces. These struts and ties meet at nodes **LRFD [5.8.2.1]**. See Figure 13.7-1 for a basic strut-and-tie model that depicts two bearing reactions transferred to two columns. STM is used to determine internal force effects at the strength and extreme event limit states.

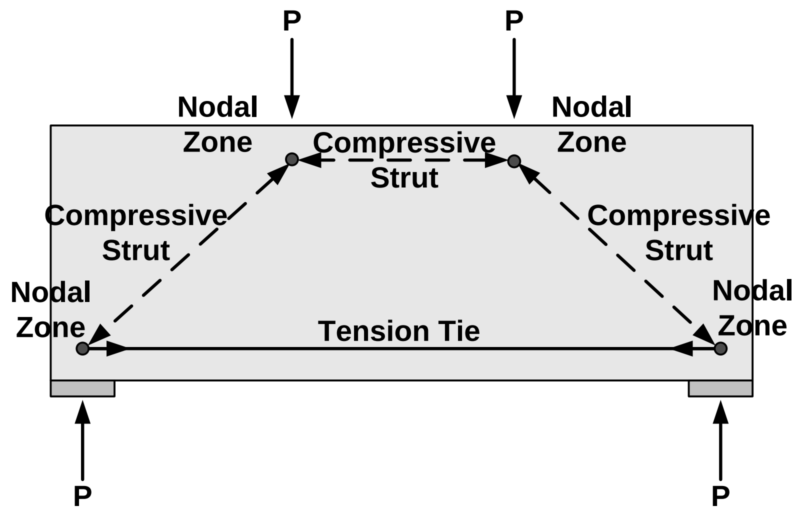

Figure 13.7-1
Basic Strut-and-Tie Elements

Strut-and-tie models are based on the following assumptions:

- The tension ties yield before the compressive struts crush.
- External forces are applied at nodes.
- Forces in the struts and ties are uniaxial.
- Equilibrium is maintained.
- Prestressing of the pier is treated as a load.

The generation of the model requires informed engineering judgment and is an iterative, graphical procedure. The following steps are recommended for a strut-and-tie pier cap design.

13.7.1 Draw the Idealized Truss Model

This model will be based on the structure geometry and loading configuration **LRFD [5.8.2.2]**. At a minimum, nodes shall be placed at each load and support point. Maintain angles of approximately 30° (minimum of 25º) to 60° (maximum of 65º) between strut and tie members that meet at a common node. An angle close to 45° should be used when possible. Figure 13.7-2 depicts an example hammerhead pier cap strut-and-tie model supporting (5) girders.

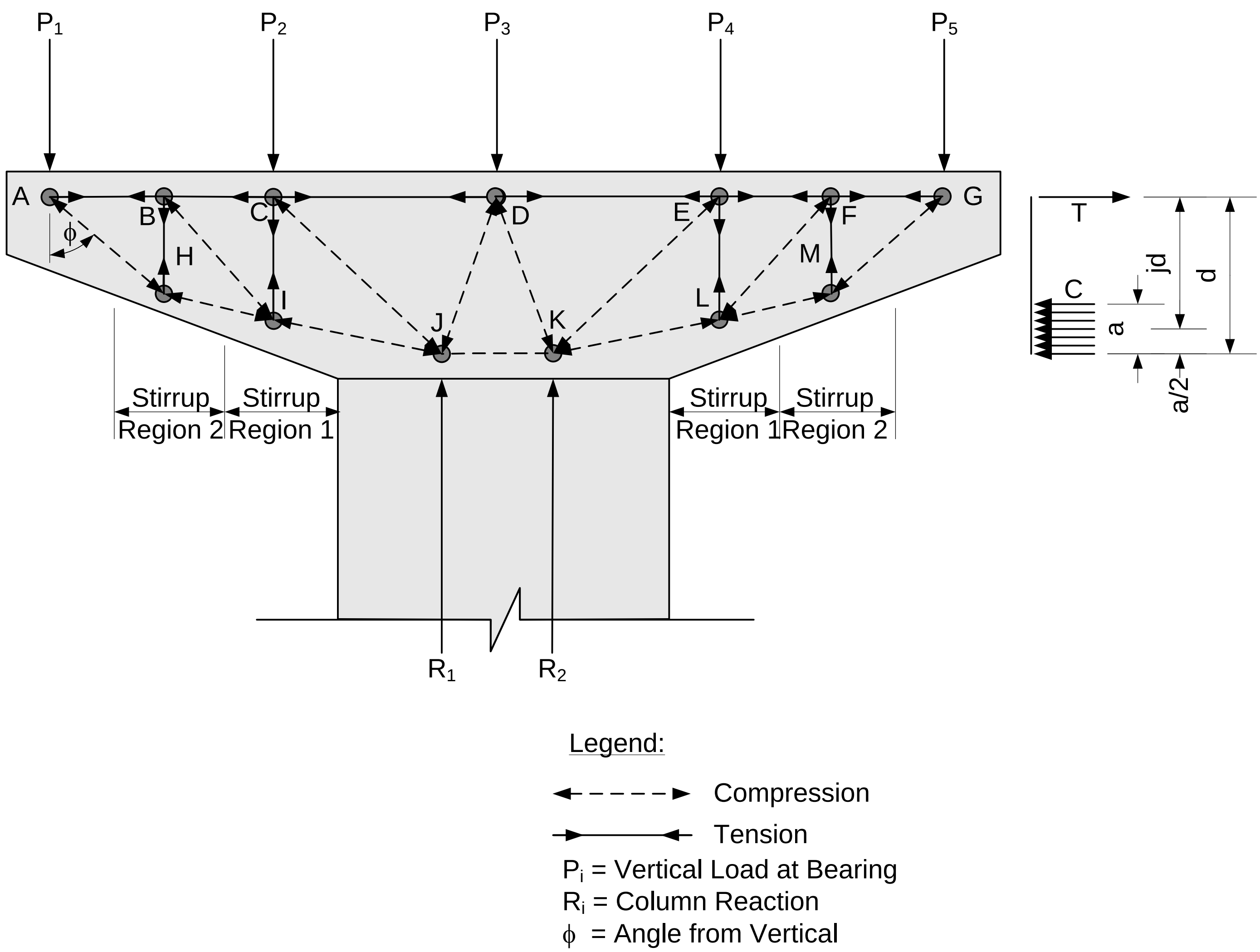

Figure 13.7-2
Example Hammerhead Pier Cap Strut-and-Tie Model

To begin, place nodes at the bearing locations and at the two column 1/3-points. In Figure 13.7-2, the minimum of nodes A, C, D, E and G are all placed at a bearing location, and nodes J and K are placed at the column 1/3-points. When drawing the truss model, the order of priority for forming compressive struts shall be the following:

1. Transfer the load directly to the column if the angle from vertical is less than 60°.

2. Transfer the load to a point directly beneath a bearing if the angle from vertical is between 30° and 60°.

3. Transfer the load at an approximately 45° angle from vertical and form a new node.

In Figure 13.7-2, the bearing load at node C is transferred directly to the column at node J since the angle formed by the compression strut C-J is less than 60°. The same occurs at strut E-K. However, the angle that would be formed by compression strut A-J to the column is not less than 60°, nor is the angle that would be formed by a strut A-I to beneath a bearing. Therefore, the load at node A is transferred at a 45° angle to node H by strut A-H. To maintain equilibrium at node H, the vertical tension tie B-H and the compression strut H-I are added.

Then, since the angle that would be formed by potential column strut B-J is not less than 60°, a check is made of the angle that would be formed by strut B-I. Since this angle is within the 30° to 60° range, compression strut B-I is added. To maintain equilibrium at node I, the vertical tension tie C-I and the compression strut I-J are added. This completes the basic strut-and-tie model for the left side of the cap. The geometric setup on the right side of the cap will be performed in the same manner as the left side.

The bearing load at node D, located above the column, is then distributed directly to the column as the angle from vertical of struts D-J and D-K are both less than 60°. Compression strut J-K must then be added to satisfy equilibrium at nodes J and K.

Vertically, the top chord nodes A, B, C, D, E, F and G shall be placed at the centroid of the tension steel. The bottom chord nodes H, I, J, K, L and M shall follow the taper of the pier cap and be placed at mid-height of the compression block, a/2, as shown in Figure 13.7-2.

The engineer should then make minor adjustments to the model using engineering judgment. In this particular model, this should be done with node H in order to make struts A-H and B-I parallel. The original 45° angle used to form strut A-H likely did not place node H halfway between nodes A and C. The angle of strut A-H should be adjusted so that node H is placed halfway between nodes A and C.

Another adjustment the engineer may want to consider would be placing four nodes above the column at 1/5-points as opposed to the conservative approach of the two column nodes shown in Figure 13.7-2 at 1/3-points. The four nodes would result in a decrease in the magnitude of the force in tension tie C-I. If the structure geometry were such that girder P_2 were placed above the column or the angle from vertical for potential strut B-J were less than 60°, then the tension tie C-I would not be present.

Proportions of nodal regions should be based on the bearing dimensions, reinforcement location, and depth of the compression zone. Nodes may be characterized as:

- CCC: Nodes where only struts intersect

- CCT: Nodes where a tie intersects the node in only one direction

- CTT: Nodes where ties intersect in two different directions

13.7.2 Solve for the Member Forces

Determine the magnitude of the unknown forces in the internal tension ties and compression struts by transferring the known external forces, such as the bearing reactions, through the strut-and-tie model. To satisfy equilibrium, the sum of all vertical and horizontal forces acting at each node must equal zero.

13.7.3 Check the Size of the Bearings

Per **LRFD [5.8.2.5]**, the concrete area supporting the bearing devices shall satisfy the following:

$P_u \leq \phi \cdot P_n$ **LRFD [5.8.2.3]**

Where:

ϕ = Resistance factor for bearing on concrete, equal to 0.70, as specified in **LRFD [5.5.4.2]**

P_u = Bearing reaction from strength limit state (kips)

P_n = Nominal bearing resistance (kips)

The nominal bearing resistance of the node face shall be:

$P_n = f_{cu} \cdot A_{cn}$ **LRFD [5.8.2.5]**

Where:

f_{cu} = Limiting compressive stress at the face of a node **LRFD [5.8.2.5.3]** (ksi)

A_{cn} = Effective cross-sectional area of the node face **LRFD [5.8.2.5.2]** (in^2)

Limiting compressive stress at the node face, f_{cu}, shall be:

$f_{cu} = m \cdot \nu \cdot f'_c$

Where:

f'_c = Compressive strength of concrete (ksi)

m = Confinement modification factor **LRFD [5.6.5]**

ν = Concrete efficiency factor (0.45, when no crack control reinforcement is present ; see **LRFD [Table 5.8.2.5.3a-1]** for values when crack control reinforcement is present per **LRFD [5.8.2.6]**)

For node regions with bearings:

$A_{cn} = A_{brg}$ = Area under bearing device (in^2)

$P_n = (m \cdot \nu \cdot f'_c) \cdot A_{brg}$; therefore $A_{brg} \geq P_u / \phi \cdot (m \cdot \nu \cdot f'_c)$

- Node regions with no crack control reinforcement:

$A_{brg} \geq P_u / \phi \cdot (m \cdot 0.45 \cdot f'_c)$

- Node regions with crack control reinforcement per **LRFD [5.8.2.6]**:

$A_{brg} \geq P_u / \phi \cdot (m \cdot 0.85 \cdot f'_c)$ --- (CCC) Node

$A_{brg} \geq P_u / \phi \cdot (m \cdot 0.70 \cdot f'_c)$ --- (CCT) Node

$A_{brg} \geq P_u / \phi \cdot (m \cdot 0.65 \cdot f'_c)$ --- (CTT) Node

Evaluate the nodes located at the bearings to find the minimum bearing area required.

13.7.4 Design Tension Tie Reinforcement

Tension ties shall be designed to resist the strength limit state force per **LRFD [5.8.2.4.1]**. For non-prestressed caps, the tension tie steel shall satisfy:

$P_u \leq \phi \cdot P_n$ **LRFD [5.8.2.3]**

$P_n = f_y \cdot A_{st}$; therefore,

$A_{st} \geq P_u / (\phi \cdot f_y)$

Where:

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

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

f_y = Yield strength of reinforcement (ksi)

P_n = Nominal resistance of tension tie (kips)

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

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 force 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 limit (L_2) of Stirrup Region 2 is 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 limit (L_1) of Stirrup Region 1 is from the column face to the midpoint between nodes B and C. Using the equations above, the minimum area of reinforcement (A_{st}) can be found for the vertical tension tie **LRFD [5.8.2.4.1]**. The number of vertical stirrup legs at a cross-section can be selected, and their total area can be calculated as ($A_{stirrup}$). The number of stirrups required will then be:

$$n = A_{st} / A_{stirrup}$$

The stirrup spacing shall then be determined by the following equation:

$$s_{max} = L_i / n$$

Where:

s_{max}	=	Maximum allowable stirrup spacing (in)
L_i	=	Length of stirrup region (in)
n	=	Number of stirrups to satisfy the area (A_{st}) required to resist the vertical tension tie force

Skin reinforcement on the side of the cap, shall be determined as per **LRFD [5.6.7]**. This reinforcement shall not be included in any strength calculations.

13.7.5 Check the Compression Strut Capacity

Compression struts shall be designed to resist the strength limit state force per **LRFD [5.8.2.5]**.

$$P_u \leq \phi \cdot P_n$$ **LRFD [5.8.2.3]**

The nominal resistance of the node face for a compression strut shall be taken as:

$$P_n = f_{cu} \cdot A_{cn}$$ **LRFD [5.8.2.5]** --- (unreinforced)

Where:

P_n = Nominal 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]**

f_{cu} = Limiting compressive stress at the face of a node **LRFD [5.8.2.5.3]** (ksi)

A_{cn} = Effective cross-sectional area of the node face at the strut **LRFD [5.8.2.5.2]** (in^2)

The limiting compressive stress at the node face, f_{cu}, shall be given by:

$$f_{cu} = m \cdot \nu \cdot f'_c$$

Where:

f'_c = Compressive strength of concrete (ksi)

m = Confinement modification factor (use m = 1.0 at strut node face)

ν = Concrete efficiency factor (<u>0.45</u>, when no crack control reinforcement is present ; see **LRFD [Table 5.8.2.5.3a-1]** for values when crack control reinforcement is present per **LRFD [5.8.2.6]**)

For node regions with struts:

$$P_n = (\nu \cdot f'_c) \cdot A_{cn} \quad \text{; therefore} \quad P_u \le \phi \cdot (\nu \cdot f'_c) \cdot A_{cn}$$

- Node regions with <u>no</u> crack control reinforcement:

$$P_u \le \phi \cdot (0.45 \cdot f'_c) \cdot A_{cn}$$

- Node regions <u>with</u> crack control reinforcement per **LRFD [5.8.2.6]**:

$P_u \le \phi \cdot (0.65 \cdot f'_c) \cdot A_{cn}$ --- (strut to node interface) --- <u>(CCC, CCT, CTT) Nodes</u>

$P_u \le \phi \cdot (0.85 \cdot f'_c) \cdot A_{cn}$ --- (back face) --- <u>(CCC) Node</u>

$P_u \le \phi \cdot (0.70 \cdot f'_c) \cdot A_{cn}$ --- (back face) --- <u>(CCT) Node</u>

$P_u \le \phi \cdot (0.65 \cdot f'_c) \cdot A_{cn}$ --- (back face) --- <u>(CTT) Node</u>

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

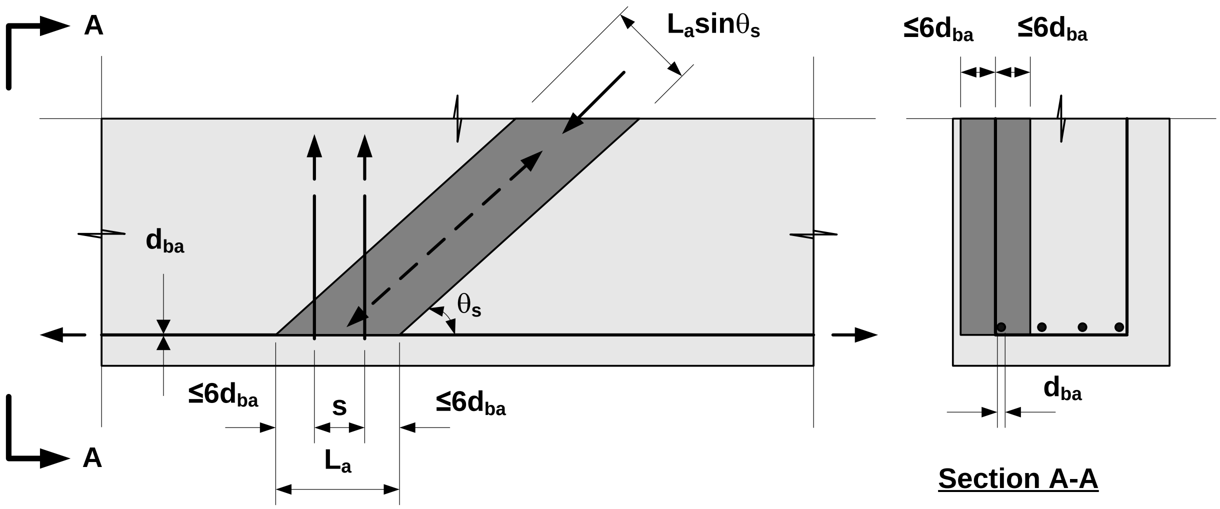

Figure 13.7-3
Strut Anchored by Tension Reinforcement Only (CTT)

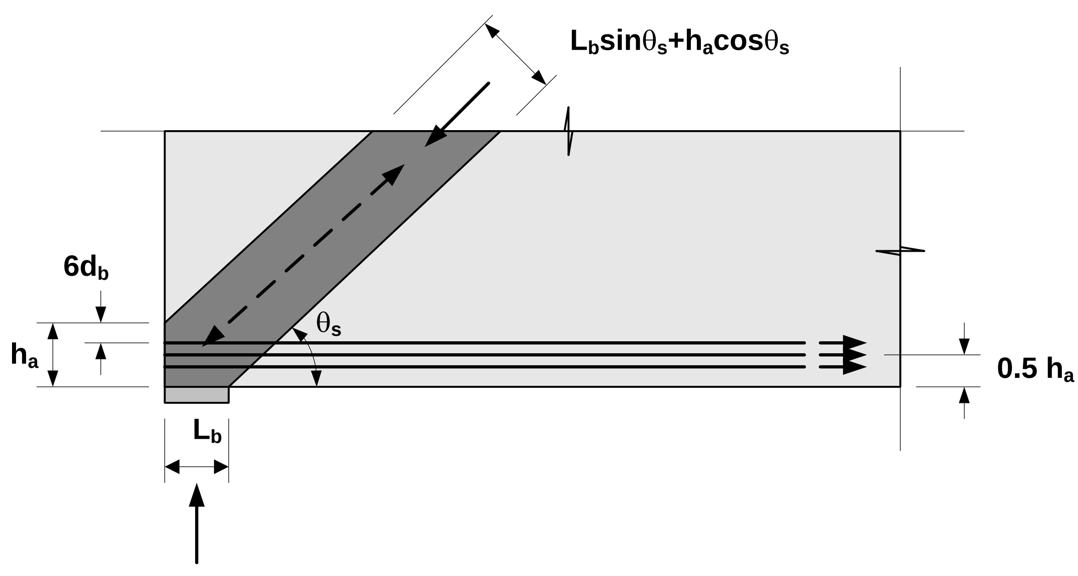

Figure 13.7-4
Strut Anchored by Bearing and Tension Reinforcement (CCT)

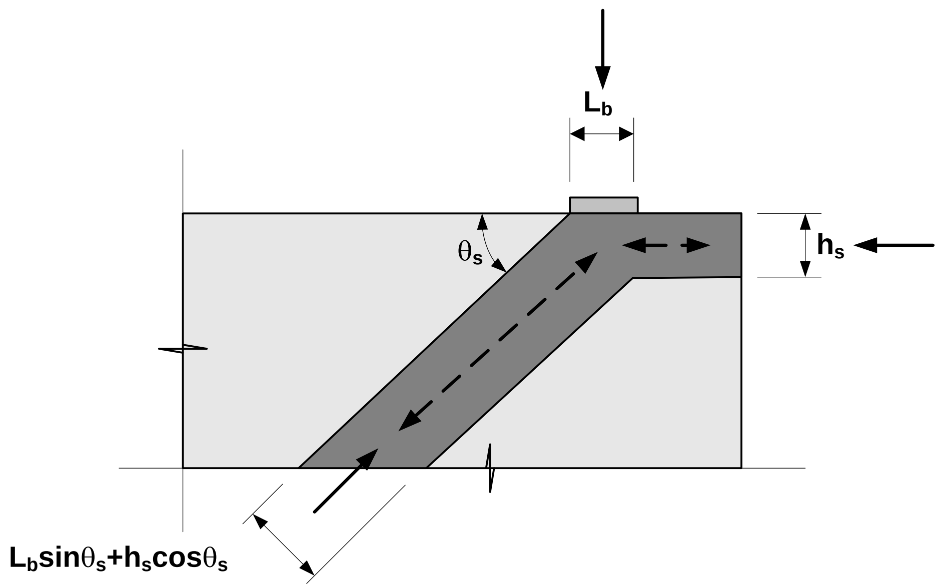

Figure 13.7-5
Strut Anchored by Bearing and Strut (CCC)

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

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

13.7.6 Check the Tension Tie Anchorage

Tension ties shall be anchored to the nodal zones by either specified embedment length or hooks so that the tension force may be transferred to the nodal zone. As specified in **LRFD [5.8.2.4.2]**, the tie reinforcement shall be fully developed at the inner face of the nodal zone. In Figure 13.7-4, this location is given by the edge of the bearing where θ_s is shown. Develop tension reinforcement per requirements specified in **LRFD [5.10.8]**.

13.7.7 Provide Crack Control Reinforcement

Pier caps designed using the strut-and-tie method and the efficiency factors of **LRFD [Table 5.8.2.5.3a-1]**, shall contain an orthogonal grid of reinforcing bars near each face in accordance with **LRFD [5.8.2.6]**. This reinforcement is intended to control the width of cracks and to ensure a minimum ductility for the member so that, if required, significant redistribution of internal stresses can take place. Crack control reinforcement shall consist of two grids distributed evenly near each side face of the strut. Additional internal layers may be used when necessary for thicker members, in order to provide a practical layout. Maximum bar spacing shall not exceed the smaller of d/4 and 12”. This reinforcement is not to be included as part of the tie.

The reinforcement in the vertical direction shall satisfy:

$$A_v / b_w \cdot s_v \geq 0.003 \quad \text{; therefore} \quad A_v \geq (0.003)\, b_w \cdot s_v$$

The reinforcement in the horizontal direction shall satisfy:

$$A_h / b_w \cdot s_h \geq 0.003 \quad \text{; therefore} \quad A_h \geq (0.003)\, b_w \cdot s_h$$

Where:

A_v = Total area of vertical crack control reinforcement within spacing s_v (in.)

A_h = Total area of horizontal crack control reinforcement within spacing s_h (in.)

b_w = Width of member (in.)

s_v, s_h = Spacing of vertical and horizontal crack control reinforcement (in.)

13.8 General Pier Cap Information

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

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

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

Where:

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

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

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

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

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

Crack control, as defined in **LRFD [5.6.7]** shall be considered for pier caps. Class 2 exposure condition exposure factors shall only be used when concern regarding corrosion (i.e., pier caps located below expansion joints, pier caps subject to intermittent moisture above waterways, etc.) or significant aesthetic appearance of the pier cap is present.

13.9 Column / Shaft Design

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

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

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

Wisconsin uses tied columns following the procedures of **LRFD [5.6.4]**. The minimum allowable column size is 2'-6" in diameter. The minimum steel bar area is as specified in **LRFD [5.6.4.2]**. For piers not requiring a certain percentage of reinforcement as per 13.4.10 to satisfy **LRFD [3.6.5]** for vehicular collision load, a reduced effective area of reinforcement may be used when the cross-section is larger than that required to resist the applied loading.

The computed column moments are to consider moment magnification factors for slenderness effects as specified in **LRFD [5.6.4.3]**. Values for the effective length factor, K, are as follows:

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

The computed moments are multiplied by the moment magnification factors, if applicable, and the column is designed for the combined effects of axial load and bending. According to **LRFD [5.6.4.1]** all force effects, including magnified moments, shall be transferred to adjacent components. The design resistance under combined axial load and bending is based on stress-strain compatibility. A computer program is recommended for determining the column's resistance to the limit state loads.

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

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

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

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

Crack control, as defined in **LRFD [5.6.7]** shall be considered for pier columns. All pier columns shall be designed using a Class 2 exposure condition exposure factor.

13.10 Pile Bent and Pile Encased Pier Analysis

WisDOT policy item:

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

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

WisDOT policy item:

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

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

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

Where:

F	=	Total factored vertical load (kips)
n	=	Number of piles
M	=	Total factored moment about pile group centroid (kip-ft)
S	=	Section modulus of pile group (ft^3), equal to:

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

In which:

d	=	Distance of pile from pile group centroid
c	=	Distance from outermost pile to pile group centroid

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

13.11 Footing Design

13.11.1 General Footing Considerations

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

For multi-columned piers:

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

For hammerhead piers:

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

WisDOT exception to AASHTO:

Crack control, as defined in **LRFD [5.6.7]** shall not be considered for pier isolated spread footings, isolated pile footings and continuous footings.

Shrinkage and temperature reinforcement, as defined in **LRFD [5.10.6]** shall not be considered for side faces of any buried footings.

13.11.2 Isolated Spread Footings

Spread footings are designed using LRFD strength limit state loads and resistance for moment and shear as specified in **LRFD [5.12.8]**. The foundation bearing capacity, used to dimension the footing's length and width, shall be determined using **LRFD [10.6]** of the AASHTO *LRFD Bridge Design Specifications*.

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

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

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

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

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

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

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

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

6. The spread footing's reinforcing steel is determined from the flexural requirements of **LRFD [5.6.3]**. The design moment is determined from the volume of the pressure diagram under the footing which acts outside of the section being considered. The weight of the footing and the soil above the footing is used to reduce the bending moment.

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

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

    a. Two-way action

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

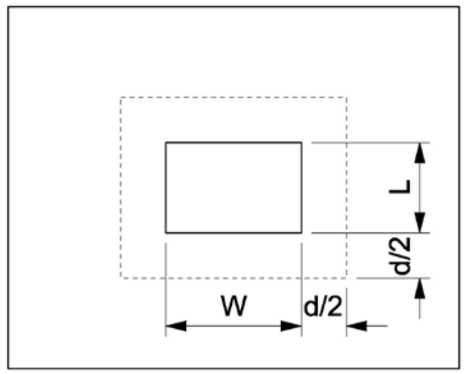

Figure 13.11-1
Critical Perimeter Location for Spread Footings

b. One-way action

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

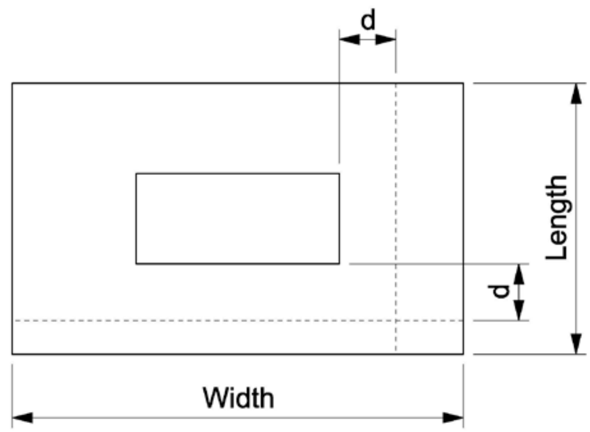

Figure 13.11-2
Shear Location for One-Way Action

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

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

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

13.11.3 Isolated Pile Footings

WisDOT policy item:

Pile footings are designed using LRFD strength limit state loads and resistance for moment and shear as specified in **LRFD [5.12.8]**. The pile design shall use LRFD strength limit state loads to compare to the factored axial compression resistance specified in Table 11.3-5.

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

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

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

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

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

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

3. Same as spread footing.

4. Same as spread footing.

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

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

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

8. The pile footing reinforcing steel is determined from the flexural requirements of **LRFD [5.6.3]**. The design moment and shear are determined from the force of the piles which act outside of the section being considered. The weight of the footing and the soil above the footing are used to reduce the magnitude of the bending moment and shear force.

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

a. Two-way action

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

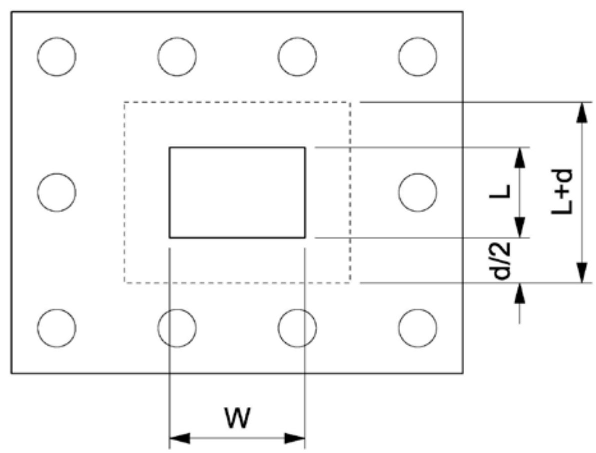

Figure 13.11-3
Critical Perimeter Location for Pile Footings

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

b. One-way action

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

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

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

13.11.4 Continuous Footings

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

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

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

13.11.5 Cofferdams and Seals

A cofferdam is a temporary structure used to construct concrete substructures in or near water. The cofferdam protects the substructure during construction, controls sediments, and can be dewatered to construct the substructure in a dry environment. Dewatering the cofferdam allows for the cutting of piles, placement of reinforcing steel and ensuring proper consolidation of concrete. A cofferdam typically consists of driven steel sheet piling and allows for the structure to be safely dewatered when properly designed. Alternative cofferdam systems may be used to control shallow water conditions.

A cofferdam bid item may be warranted when water is expected at a concrete substructure unit during construction. The cofferdam shall be practically watertight to allow for dewatering such that the substructure is constructed in a dry environment. An exception is for pile encased piers with expected water depths of 5 feet or less. These substructures may be poured underwater, but in certain cases may still require a cofferdam for protection and/or to address environmental concerns. A pile encased pier with expected water depths greater than 5 feet will typically require a cofferdam. The designer should consult with geotechnical and regional personnel to determine if a cofferdam is required. If a cofferdam is warranted, then include the bid item “Cofferdams (Structure)”.

Environmental concerns (specifically sediment control) may require the use of cofferdams at some sites. When excavation takes place in the water, some form of sediment control is usually required. The use of simple turbidity barrier may not be adequate based on several considerations (water depth, velocity, soil conditions, channel width, etc.). All sediment control devices, such as turbidity barrier, shall not be included in structure plans. Refer to Chapter 10 of the FDM for erosion control and storm water quality information.

A seal is a mat of unreinforced concrete poured under water inside a cofferdam. The seal is designed to withstand the hydrostatic pressure on its bottom when the water above it is removed. For shallow water depths and certain soil conditions a concrete seal may not be necessary in order to dewater a cofferdam. Coordinate with geotechnical personnel to determine if a concrete seal is required. The designer shall determine if a concrete seal is required for a cofferdam. If a concrete seal is required, then include the bid item “Concrete Masonry Seal” and required seal dimensions. The cofferdam design shall be the responsibility of the contractor.

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

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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].** Only loads applicable for each specific wall type should be considered in the engineering analyses.

14.4.5.4 Earth Pressure

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

Active Earth Pressure

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

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

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

For walls using Coulomb earth pressure theory:

$$K_a = \frac{\sin^2(\theta + \phi'_f)}{\Gamma[\sin^2\theta \sin(\theta - \delta)]}$$ **LRFD [Eq'n 3.11.5.3-1]**

Where:

Γ = $\left[1 + \sqrt{\frac{\sin(\phi'_f + \delta)\sin(\phi'_f - B)}{\sin(\theta - \delta)\sin(\theta + B)}}\right]^2$

δ = Friction angle between fill and wall (degrees)

B = Angle of fill to the horizontal (degrees)

θ = Angle of back face of wall to the horizontal (degrees)

ϕ'_f = Effective angle of internal friction (degrees)

Note: refer to Figure 14.4-1 for details.

For walls using Rankine earth pressure theory:

$$K_a = \tan^2(45 - \phi'_f)$$

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.

For walls (normally consolidated soils, vertical wall, and level ground) using at-rest earth pressure:

$$K_o = 1 - \sin\phi'_f$$ **LRFD [Eq'n 3.11.5.2-1]**

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 equipment. 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

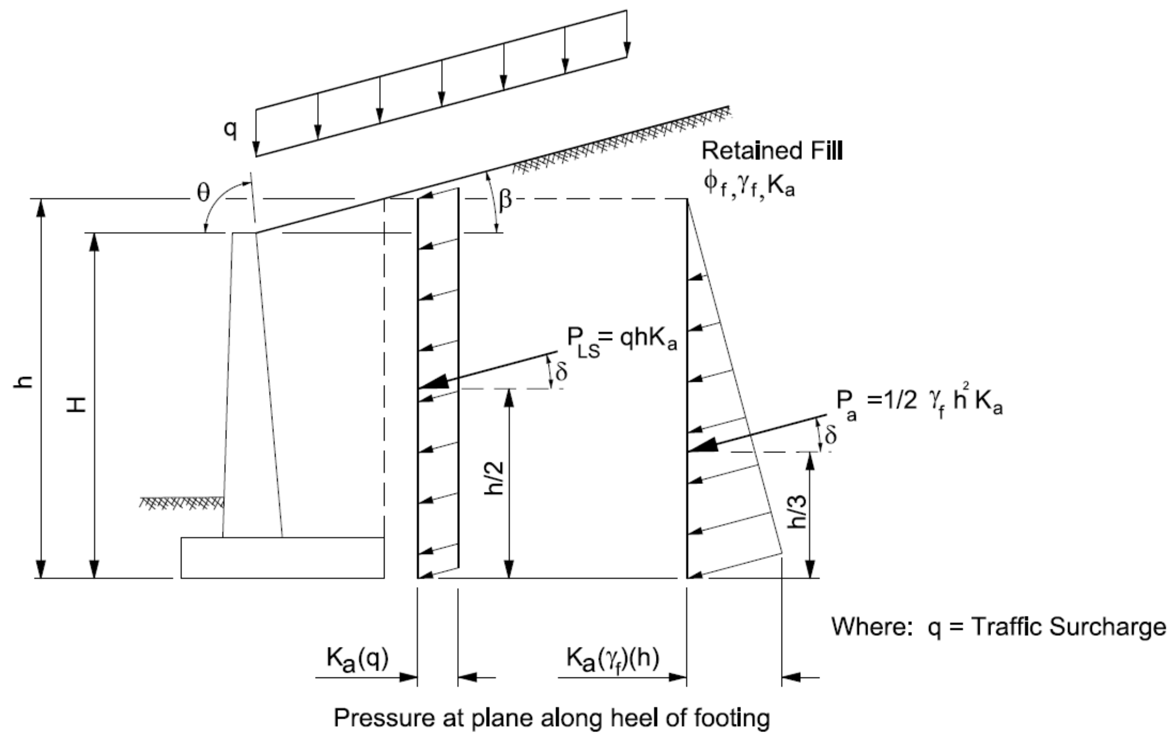
Loading diagrams for CIP cantilever, MSE, modular block gravity, and prefabricated modular walls are shown for illustration. The designer shall develop loading diagrams as applicable.

<u>CIP cantilever wall with sloping surcharge</u>

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.



<u>Figure 14.4-1</u>
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 [11.10.5.2]** 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 Rankine's theory as discussed in 14.4.5.4.

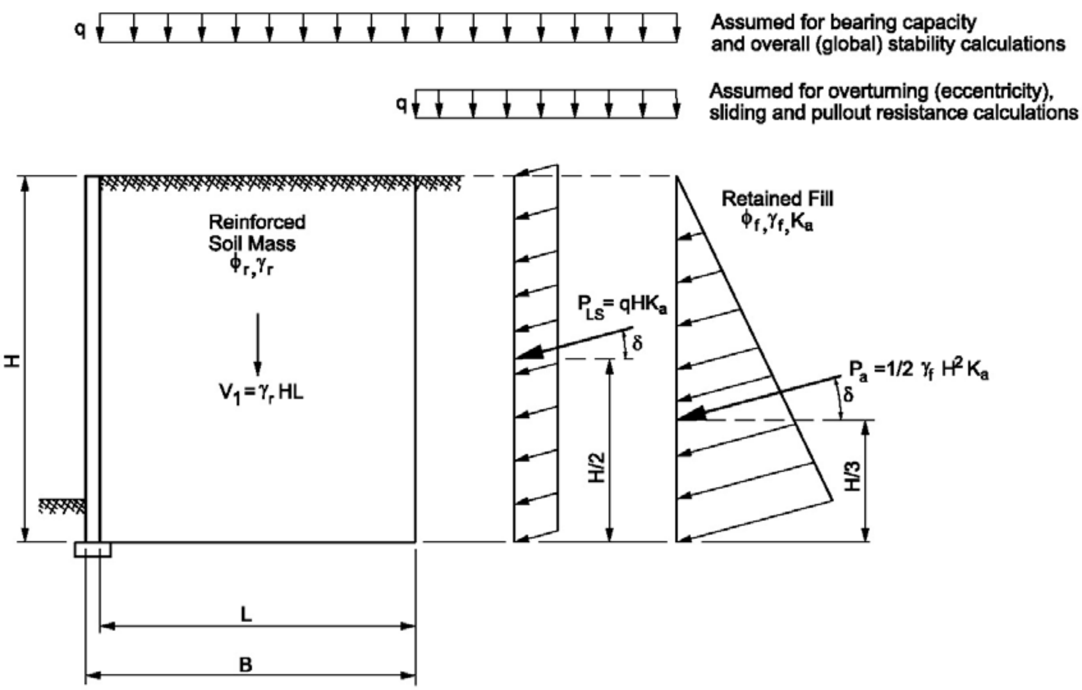


Figure 14.4-2
MSE Walls Earth Pressure for Horizontal Backslope with Traffic Surcharge
(Source **LRFD [Figure 11.10.5.2-1]**)

MSE Wall with Sloping Surcharge

Figure 14.4-3 shows a procedure to estimate the earth pressure. The active earth pressure for sloping backfill is computed using Coulomb's theory as discussed in 14.4.5.4.

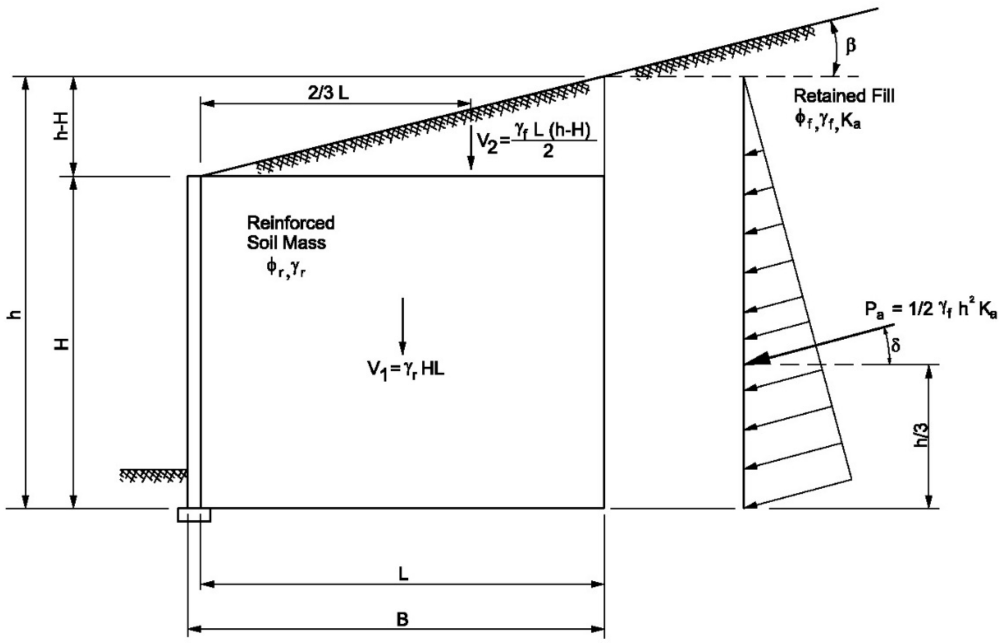


Figure 14.4-3
MSE Walls Earth Pressure for Sloping Backfill
(Source **LRFD [Figure 3.11.5.8.1-2]**)

MSE Wall with Broken Backslope

For broken backslopes, the active earth pressure coefficient is determined using Coulomb's equation except that surcharge angle β is substituted with slope angle β'.

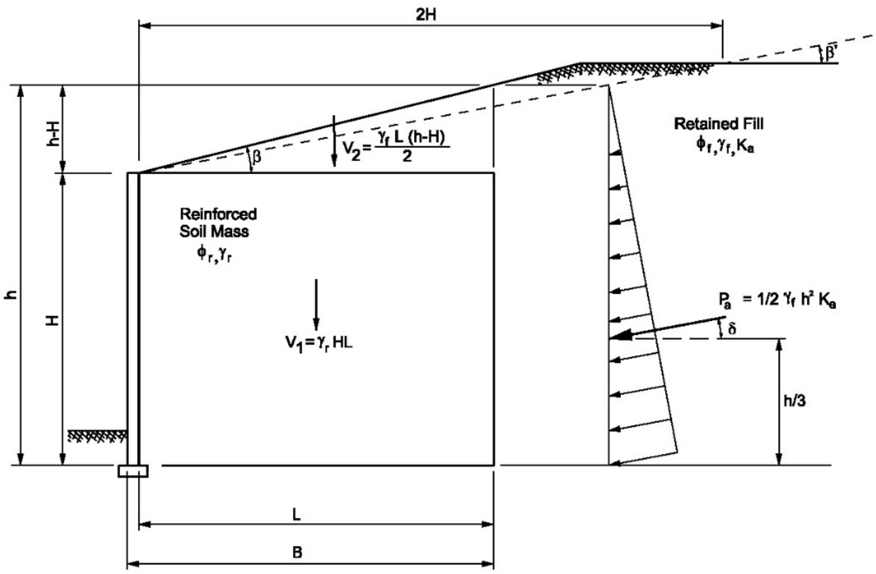


Figure 14.4-4
MSE Walls Earth Pressure for Broken Backfill
(Source **LRFD [Figure C3.11.5.8.1-1]**)

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 as discussed in 14.4.5.4.

When designing a "Modular Block Gravity Wall" with setback, the active earth pressure coefficient K_a shall be determined using Coulomb theory as discussed in 14.4.5.4. The interface friction angle between the blocks and soil behind the blocks is assumed to be zero.

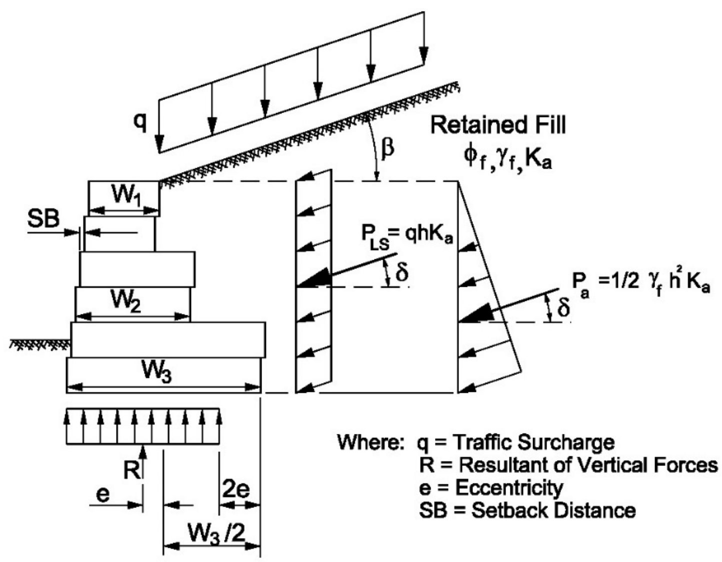


Figure 14.4-5
Modular Block Gravity Wall Analysis

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).

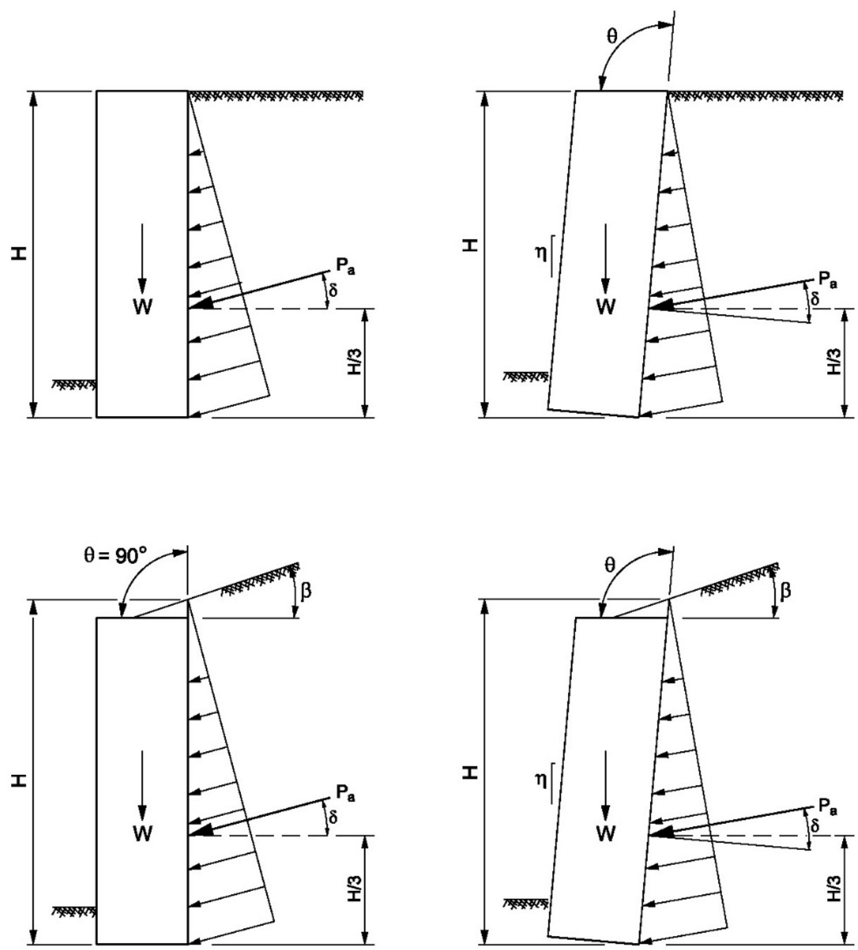


Figure 14.4-6
Lateral Earth Pressure on Concrete Modular Systems of Constant Width
(Source **LRFD [Figure 3.11.5.9-1]**)

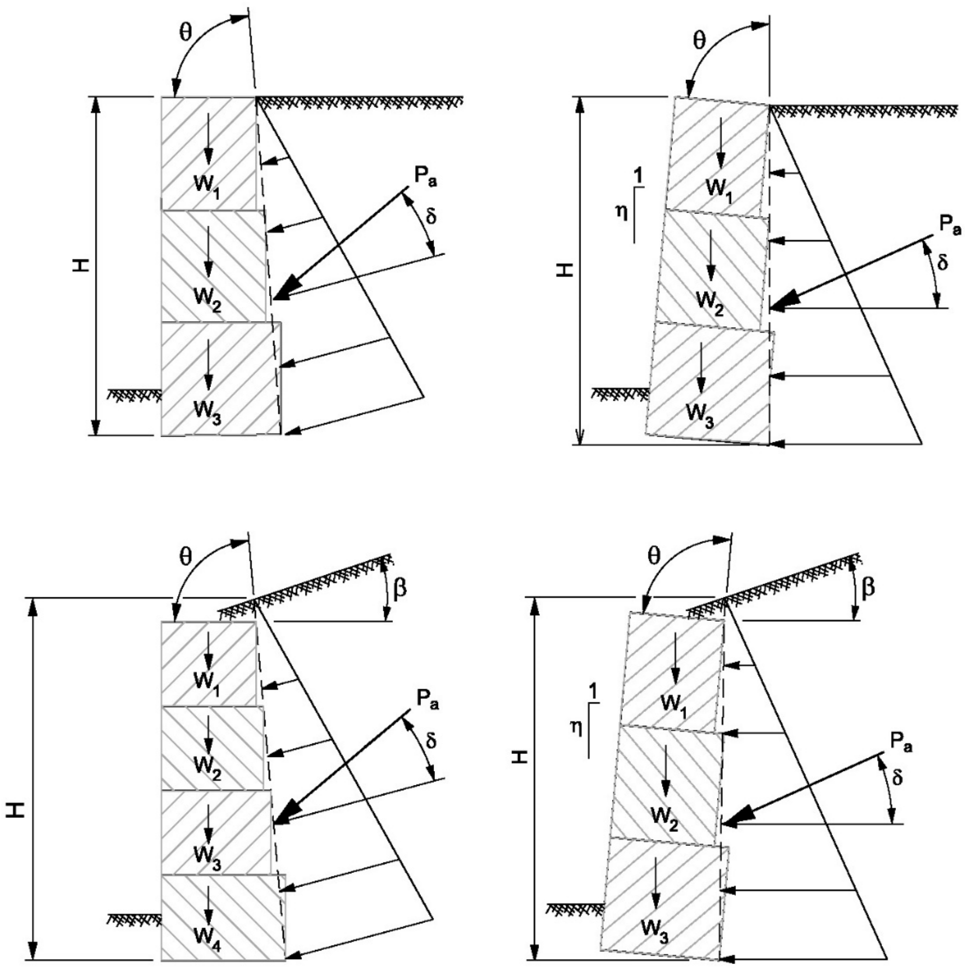


Figure 14.4-7
Lateral Earth Pressure on Concrete Modular Systems of Variable Width
(Source **LRFD [Figure 3.11.5.9-2]**)

14.4.5.5 Load factors and Load Combinations

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

Direction of Load	Load Type	Load Factor, γ_i		
		Strength I Limit		Service I Limit
		Maximum	Minimum	
Load Factors for Vertical Loads	Dead Load of Structural Components and Non-structural attachments DC	1.25	0.90	1.00
	Earth Surcharge Load ES	1.50	0.75	1.00
	Vertical Earth Load EV	1.35	1.00	1.00
	Water Load WA	1.00	1.00	1.00
	Live Load Surcharge LS	1.75	0.0	1.00
	Dead Load of Wearing Surfaces and Utilities DW	1.50	0.65	1.00
Load Factors for Horizontal Loads	Horizontal Earth Pressure EH Active At-Rest Passive	 1.50 1.35 1.35	 0.90 0.90 NA	 1.00 1.00 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]** and **LRFD [11.5.6].** 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.

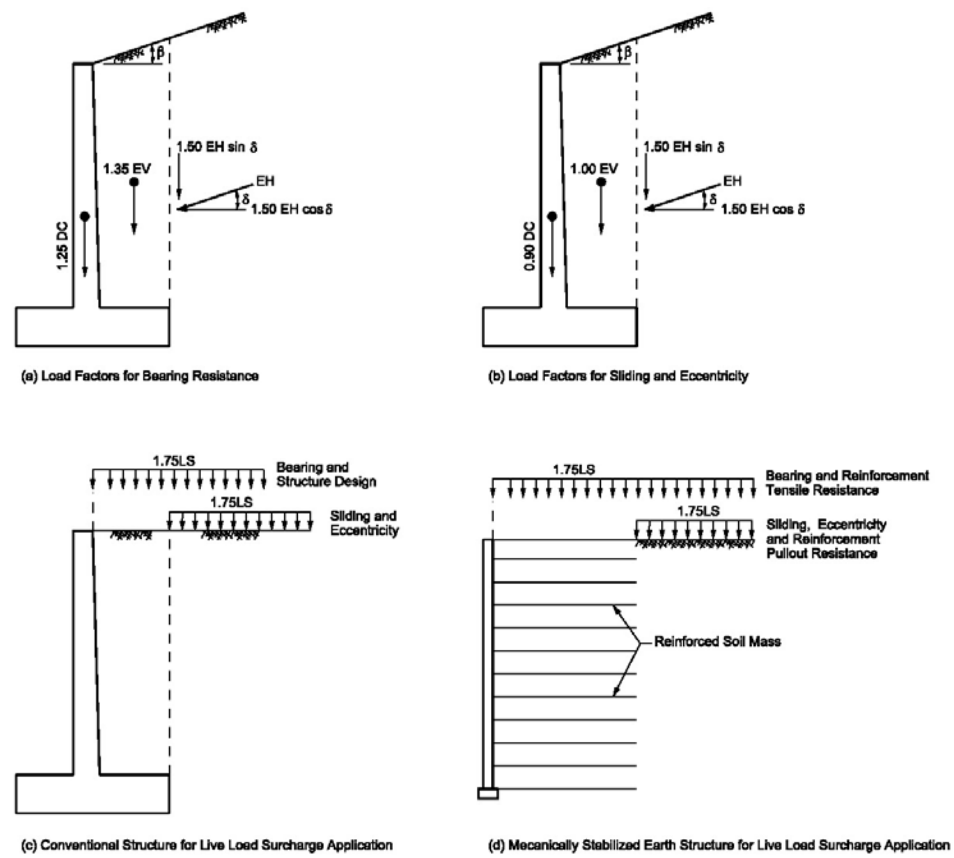


Figure 14.4-8
Application of Load Factors
(Source **LRFD [11.5.6]**)

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, 11.5.7-1]**. 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

Table 14.4-2
Resistance Factors
(Source **LRFD [Table 11.5.7-1]**)

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.

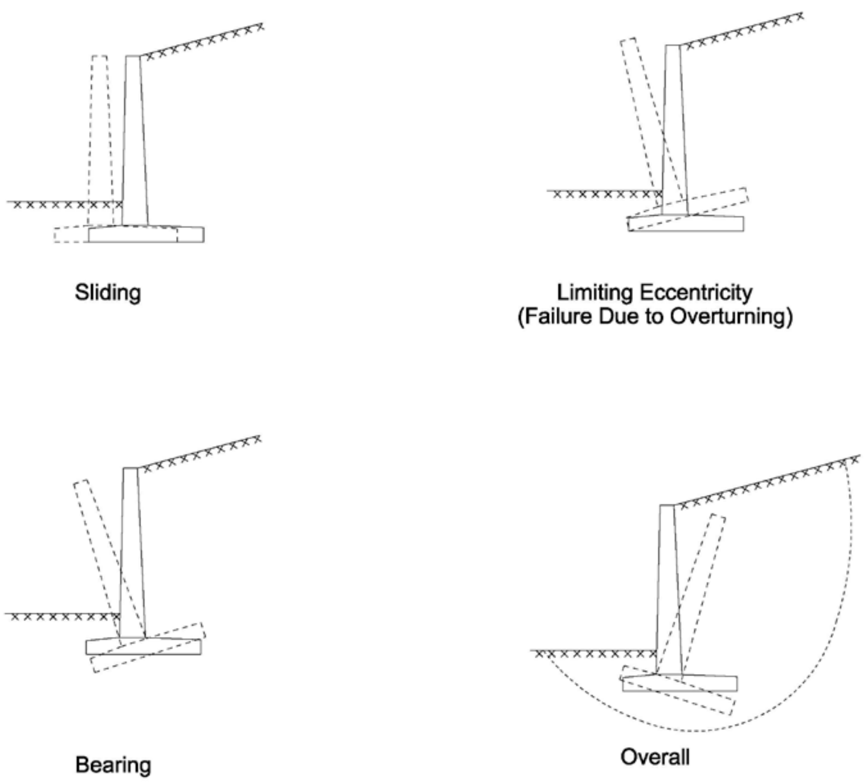


Figure14.4-9
External Stability Failure of CIP Semi-Gravity Walls

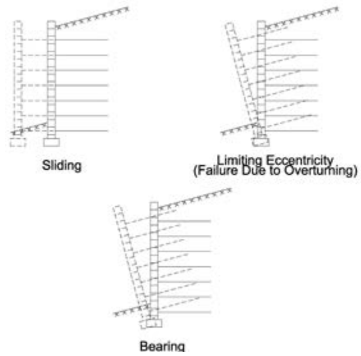


Figure 14.4-10
External Stability Failure of MSE Walls

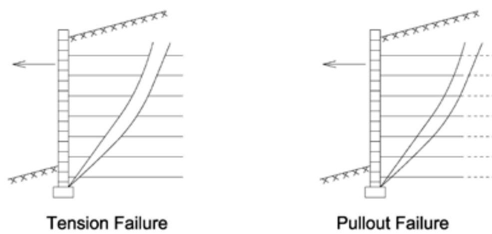


Figure 14.4-11
Internal Stability Failure of MSE Walls

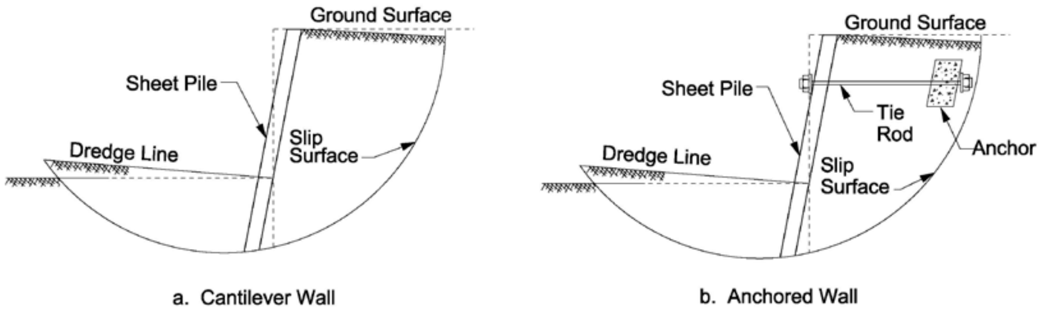


Figure 14.4-12
Deep Seated Failure of Non-Gravity Walls

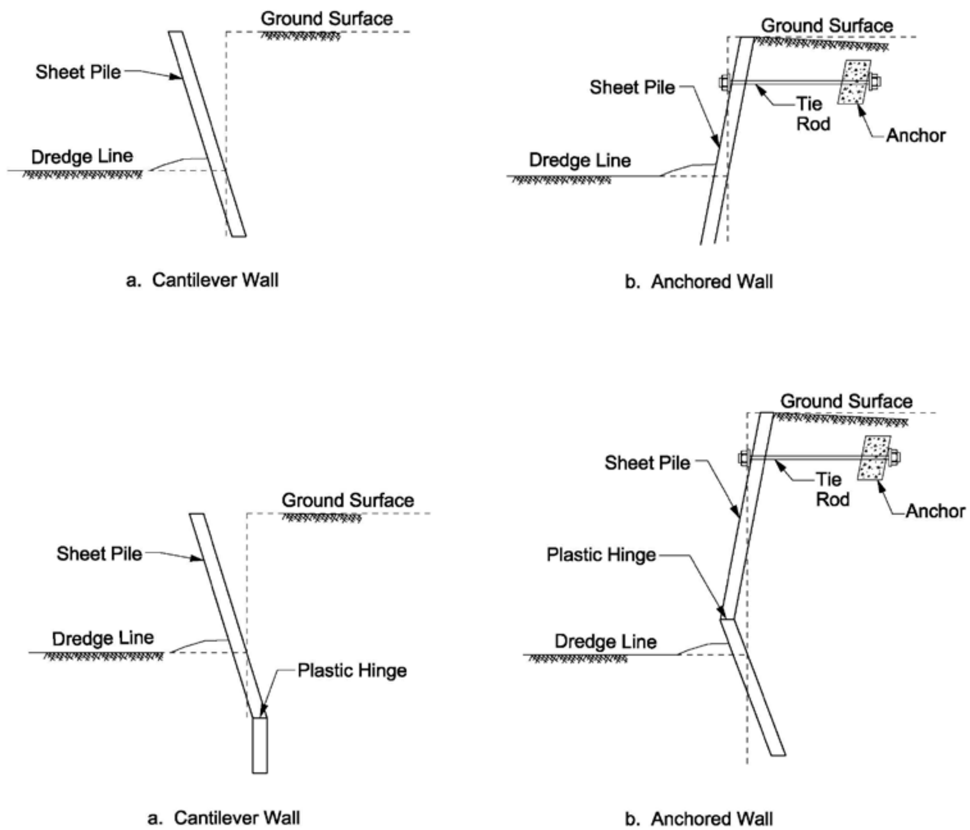


Figure 14.4-13
Flexural Failure of Non-Gravity Walls

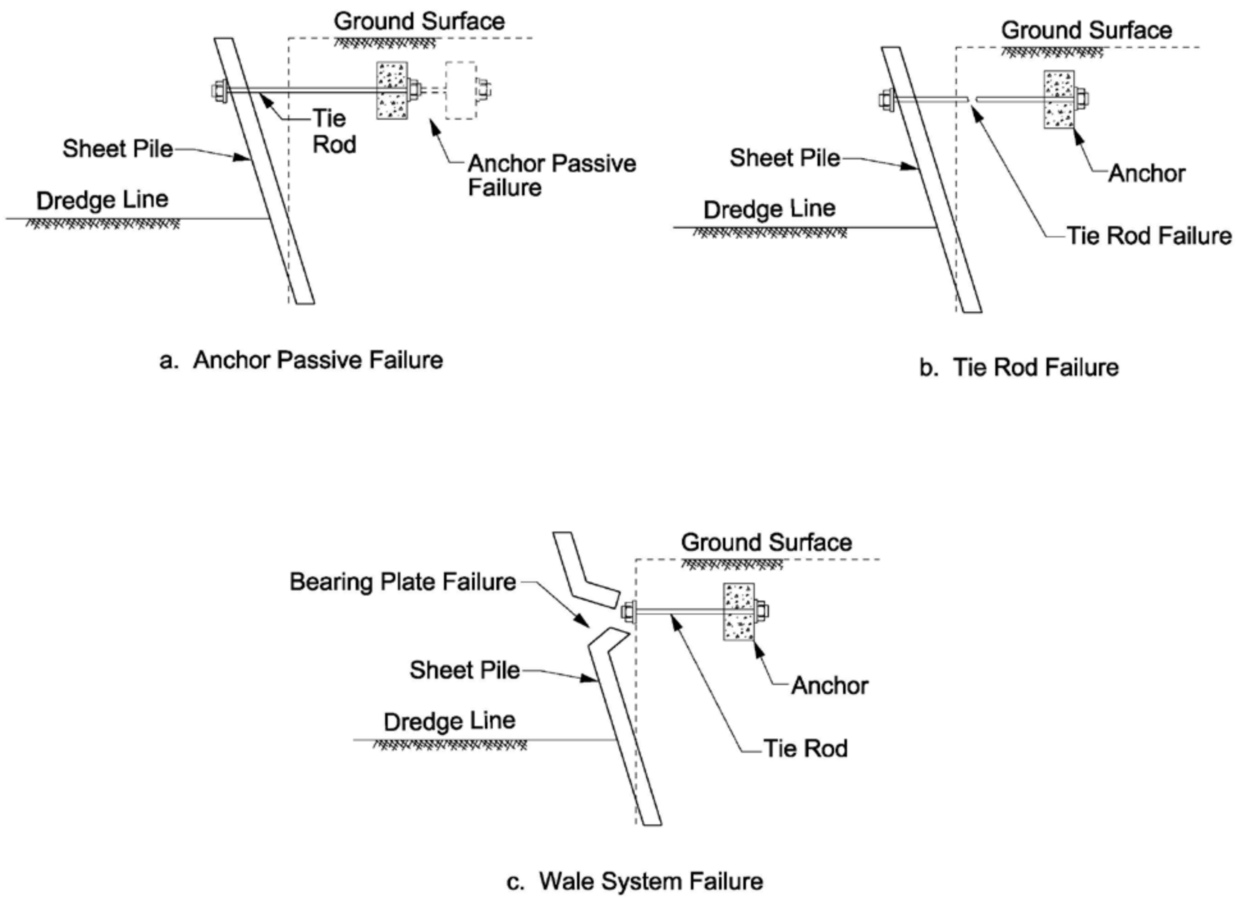


Figure 14.4-14
Flexural Failure of Non-Gravity Walls

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 modular block facing	2-4	1:200
MSE walls with geotextile /welded-wire facing	4-8	1:50-1:60
Modular block gravity walls	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

Δh1: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, 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 founded 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]**, **[11.9.7]** and **[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.

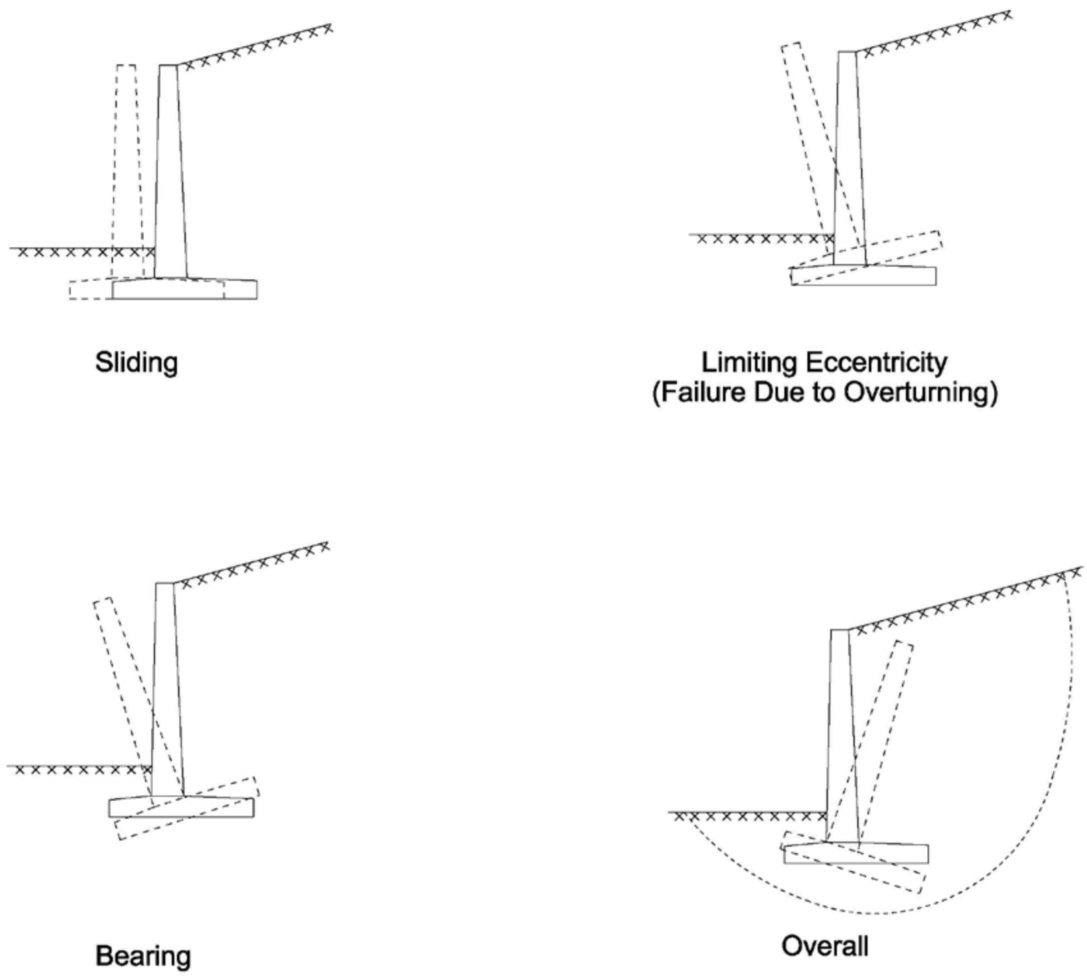


Figure 14.5-1
CIP Semi-Gravity Wall Failure Mechanism

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

3. Develop soil strength parameters

4. Determine preliminary sizing for external stability evaluation

5. Determine applicable unfactored or nominal loads

6. Evaluate factored loads for all appropriate limit states

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

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

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

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

14.5.3 Preliminary Sizing

A preliminary design can be performed using the following guideline.

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

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

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

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

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

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

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

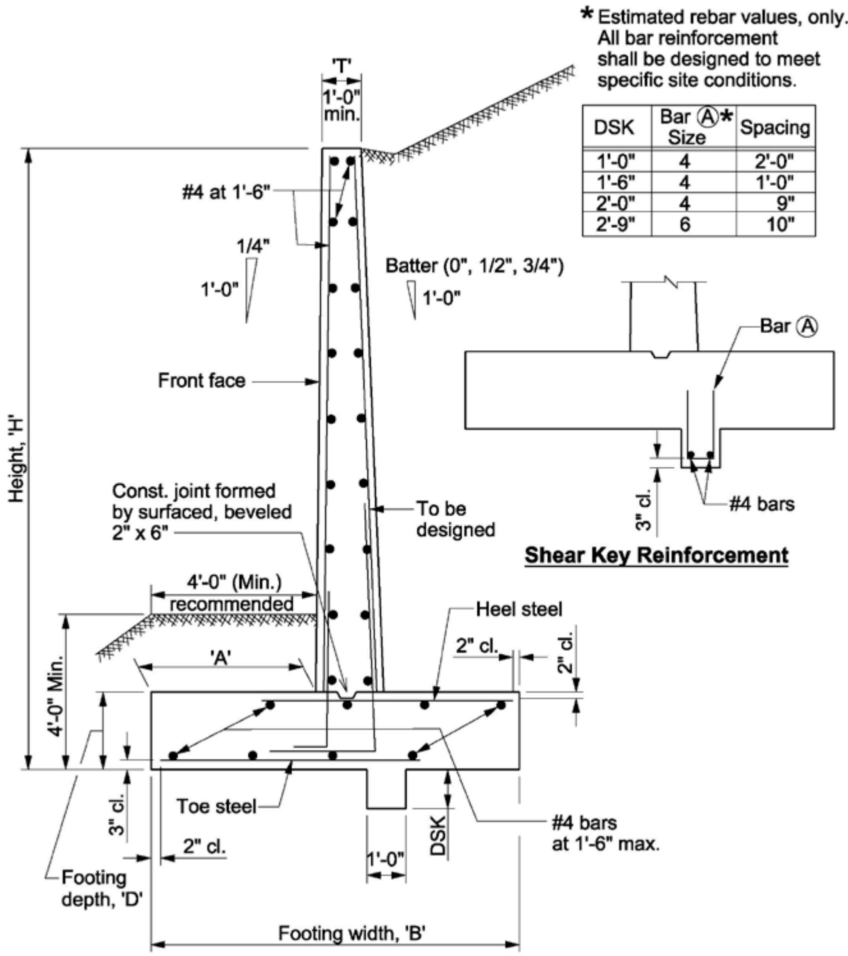


Figure 14.5-2
CIP Walls General Details

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 Figure **14.4-1** 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 eccentricity of the retaining wall shall be evaluated in accordance with **LRFD [11.6.3.3]**. The location of the resultant force should be within 1/3 of base width of the foundation centroid (e<B/3) for foundations on soil, and within 0.45 of the base width of the foundation centroid (e<0.45B) 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:

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

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:

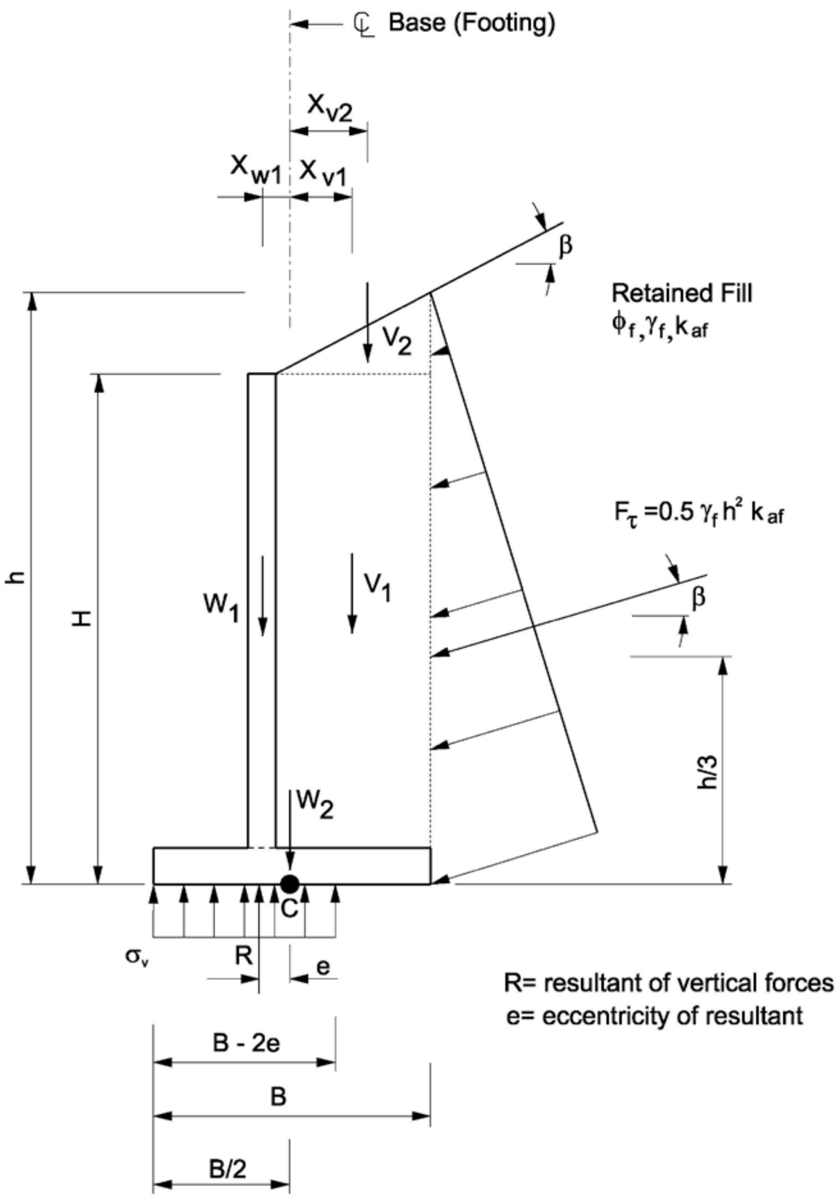
q_r = Factored bearing resistance

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

σ_v = Vertical stress

B = Base width

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

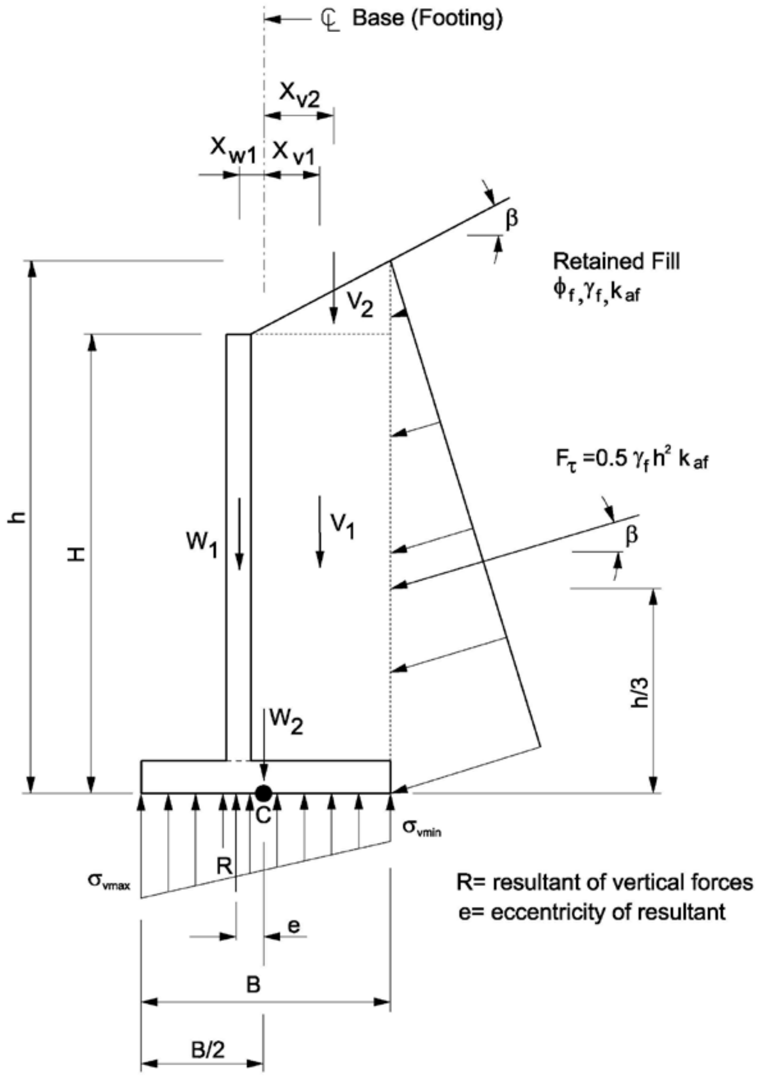


Summing Moments about Point C:

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

Figure 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:

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

Figure 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.

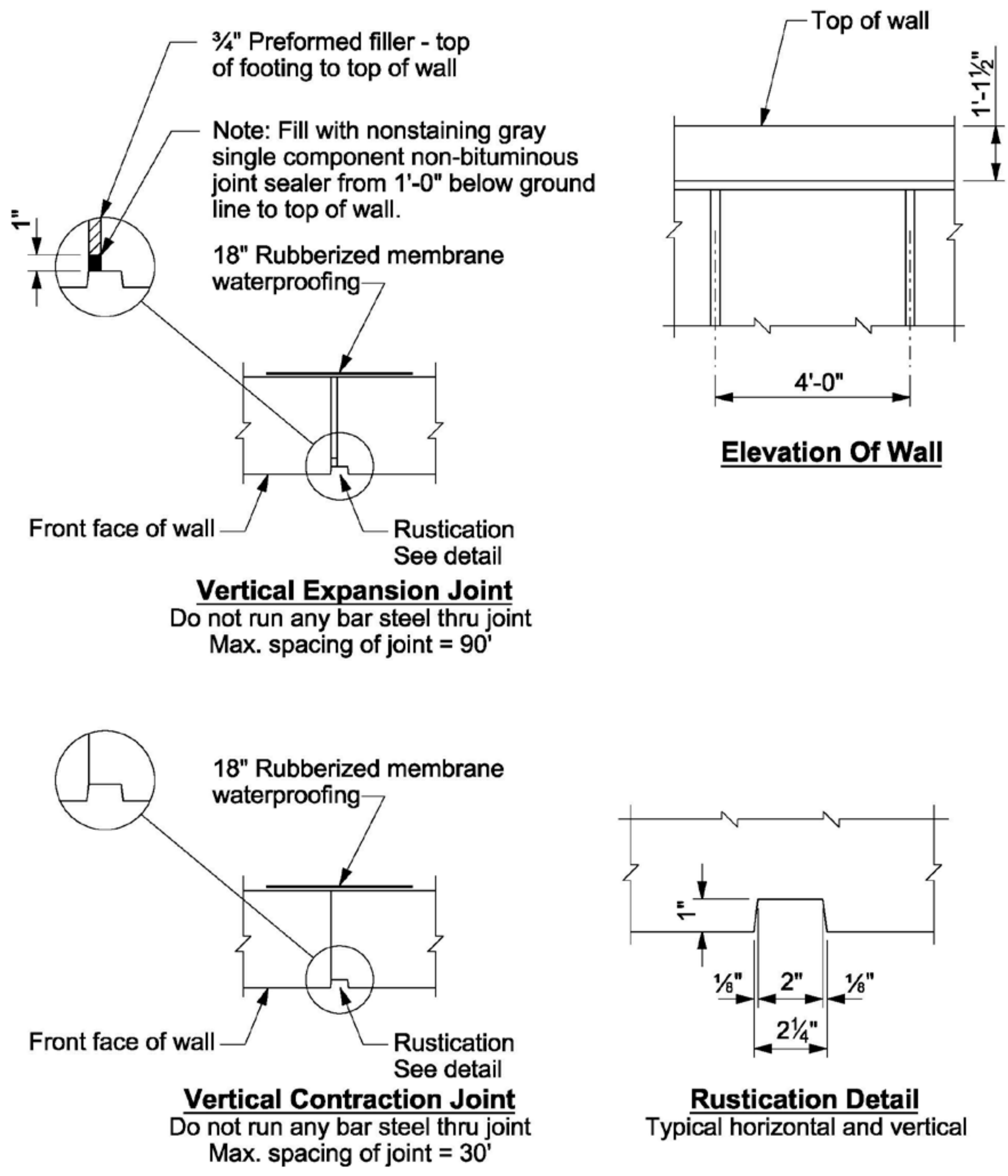


Figure 14.5-5
Retaining Wall Joint Details

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 permeable material or other positive drainage systems, placed at suitable intervals and elevations. Structure 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 (ft)	B (ft)	A (ft)	D (ft)	Batter (in/ft)	Toe Steel Size	Spa	L	Heel Steel Size	Spa	L	Stem Steel Size	Spa	Shear 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 (ft)	B (ft)	A (ft)	D (ft)	Batter (in/ft)	Toe Steel Size	Spa	L	Heel Steel Size	Spa	L	Stem Steel 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 (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
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 (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
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
      - Bearing Stress
      - Sliding

   b. Service I limit states

      - Overall Stability
      - Settlement

2. Foundation Design Parameters

   Use values provided by Geotechnical analysis

3. Concrete Design Data

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

4. Retained Soil

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

5. Soil Pressure Theory

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

6. Surcharge Load

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

7. Load Factors

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

Table 14.5-8
Load Factor Summary for CIP Walls

8. Bearing Resistance Factors

- ϕ_b = 0.55 **LRFD [Table 11.5.7-1]**

9. Sliding Resistance Factors

- ϕ_τ = 1.0 **LRFD [Table 11.5.7-1]**
- ϕ_{ep} = 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 global stability shall also be considered as part of design evaluation. MSE walls are proprietary wall systems and the design responsibilities with respect to global, external, and internal stability as well as settlement are shared between the designer (WisDOT or Consultant) and contractor. The designer is responsible for the overall stability, preliminary external stability and settlement whereas the contractor is responsible for the internal stability, compound stability and structural design of the wall. The responsibilities of the designer and contractor are outlined in 14.6.3.2. The design and drawings 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.

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:

- Selected Earthfill in the Reinforced Earth Zone
- 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.

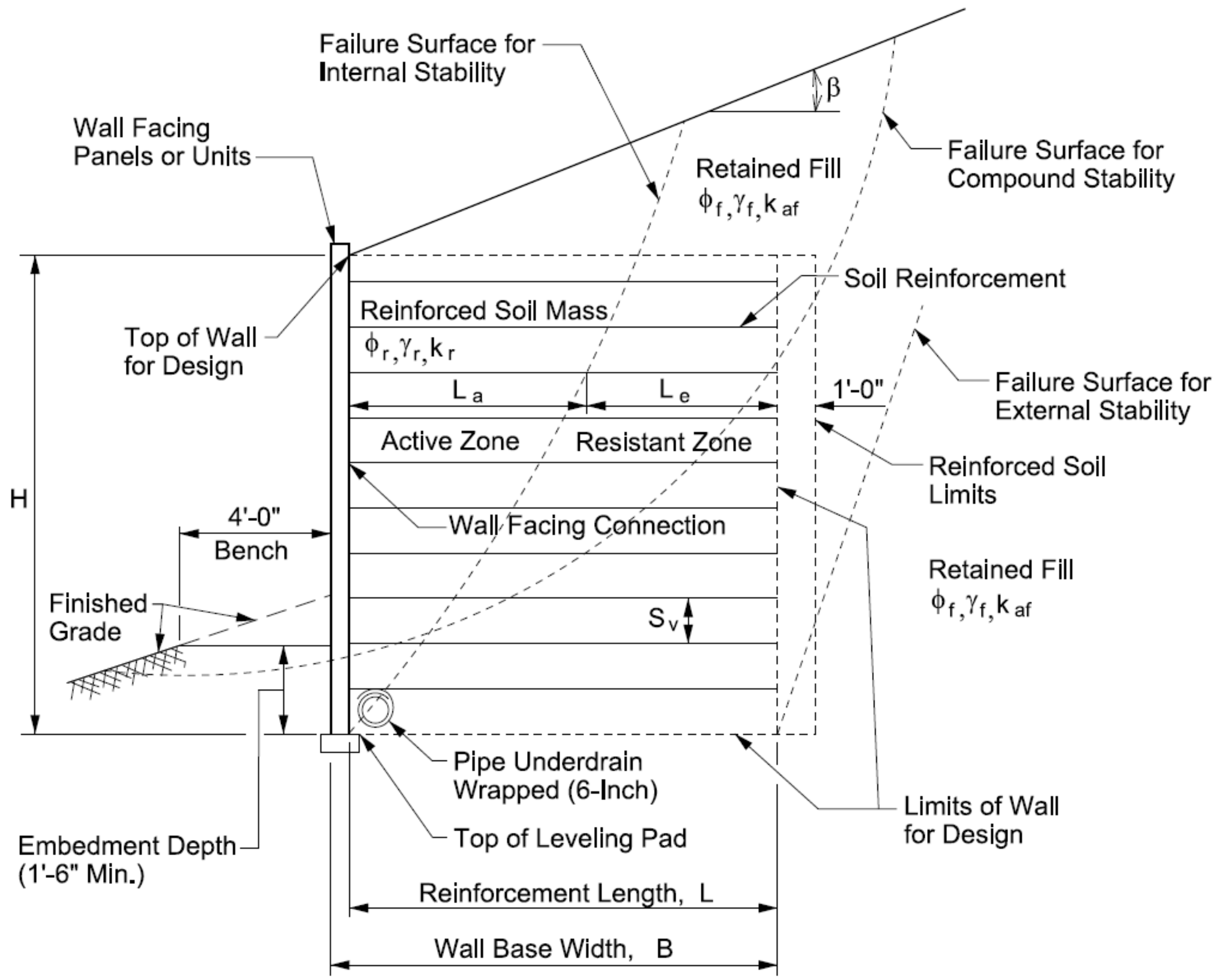


Figure 14.6-1
Structural Components of MSE Walls

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
Metallic	Chlorides	< 100 ppm
Metallic	Sulfates	< 200 ppm
Metallic	pH	5.0 < pH < 10.0
Geosynthetic	pH	4.5 < pH < 9.0
Metallic/Geosynthetic	Organic Content	< 1.0 %

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 most current wall specifications.

14.6.2.2 Reinforcement:

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

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

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

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

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

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

The non-metallic or extensible reinforcement includes the following:

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

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

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

14.6.2.3 Facing Elements

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

Major facing types are:

- Segmental precast concrete panels
- Dry cast or wet cast modular blocks
- Full height pre-cast concrete panels (tilt-up)
- 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.

Walls with curved alignments shall limit radii to 50 feet for 5 feet wide panels and 100 feet for 10 feet wide panels. Typical joint openings are not suitable for wall alignments following a tighter curve. Special joints or special panels that are less than 5 feet wide may be able to accommodate tighter curves. In general, MSE wall structures with panel type facings shall be limited to wall heights of 33 feet. Contact Structures Design Section for approval on case by case basis.

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 (inside radius) or convex (outside radius). 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. Most modular block MSE walls are reinforced with geogrids.

Modular blocks can be either dry cast or wet cast. Dry cast (small) blocks are mass produced by using a zero slump concrete that allows forms to be stripped faster than wet cast (large) blocks. MSE walls usually use dry cast blocks since they are usually a cheaper facing and wall stability is provided by the reinforced mass. Gravity walls rely on facing size and mass for wall stability. For minor walls dry cast blocks are typically used and for taller gravity walls wider wet cast blocks are normally required to satisfy stability requirements.

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.

Alignments that are not straight (i.e. kinked or curved) shall use 90 degree corners or curves. The minimum radius should be limited to 8 feet. For a concave wall the radius is measured to the front face of the bottom course. For convex walls the radius is measured to the front face of the top course. In no case shall the radius be less than 6 feet. 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).

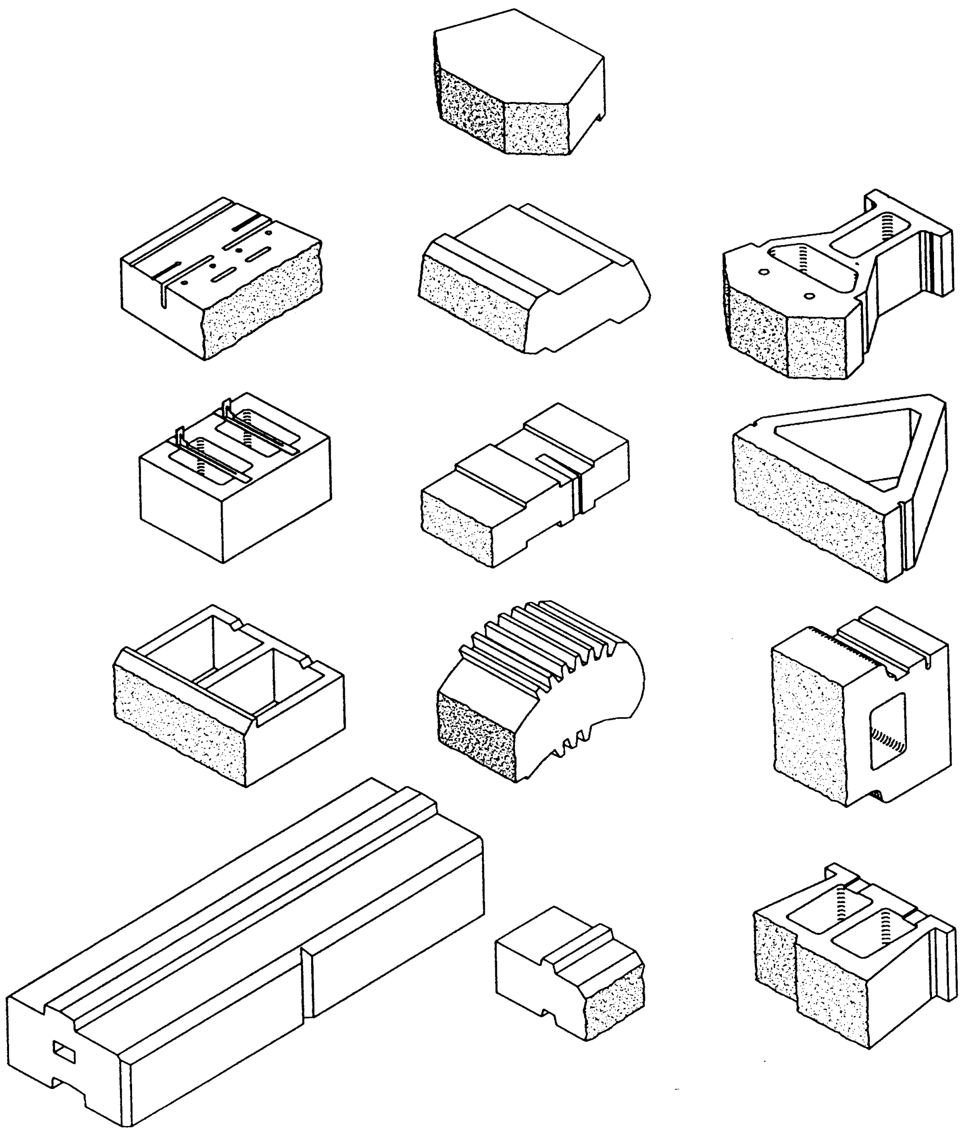
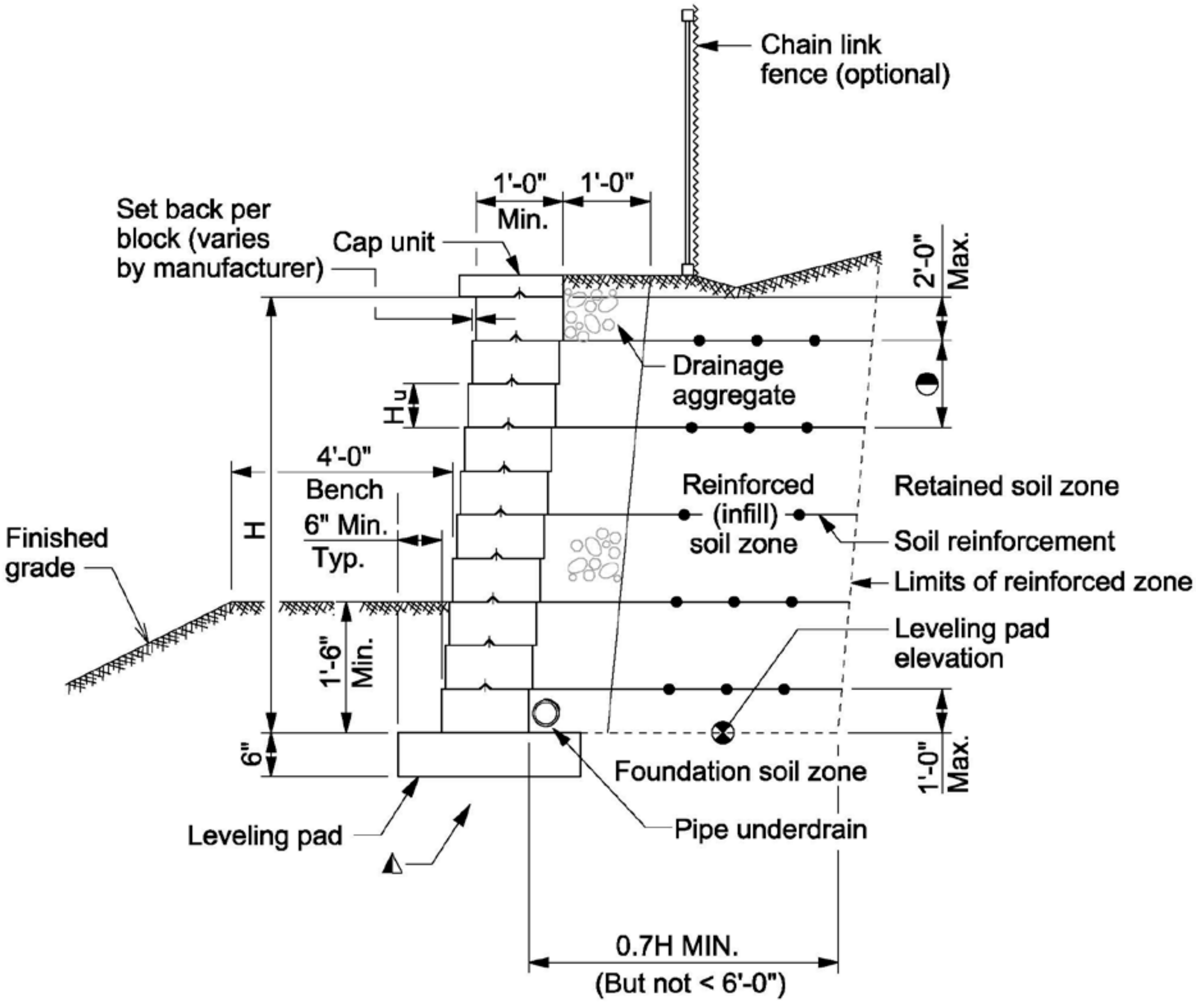


Figure 14.6-2
Modular Blocks
(Source FHWA-NHI-10-025)



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 and geotextile 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. This type of wall works well when a permanent wall facing can be placed after settlement/movement has occurred.

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 or total 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 Bureau of Structures for acceptance.

External stability, overall stability and settlement calculations are the responsibility of the WISDOT/Consultant designer. Compound 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 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 [C11.10.2.1]** 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 11.5.7-1]**; 1.0
V	=	Factored vertical dead load
δ	=	Friction angle between foundation and soil
ρ	=	Maximum soil reinforcement interface angle **LRFD [11.10.5.3]**
$\tan\delta$	=	$\tan \phi_{fd}$ 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 [11.6.3.3]** 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 two-thirds of the base width for soil foundations (i.e., e_{max} = B/3) and middle nine-tenths of the base width for rock foundations (i.e., e_{max} = 0.45B). 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 [10.6.3.2]** 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, σ_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 11.5.7-1]**.

The computed vertical stress, σ_v, shall be compared with factored bearing resistance, q_r in accordance with the **LRFD [11.10.5.4]** 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 11.5.7-1]**
CDR	=	$q_r/\sigma_v > 1.0$

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 C11.10.4-1]**.

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.

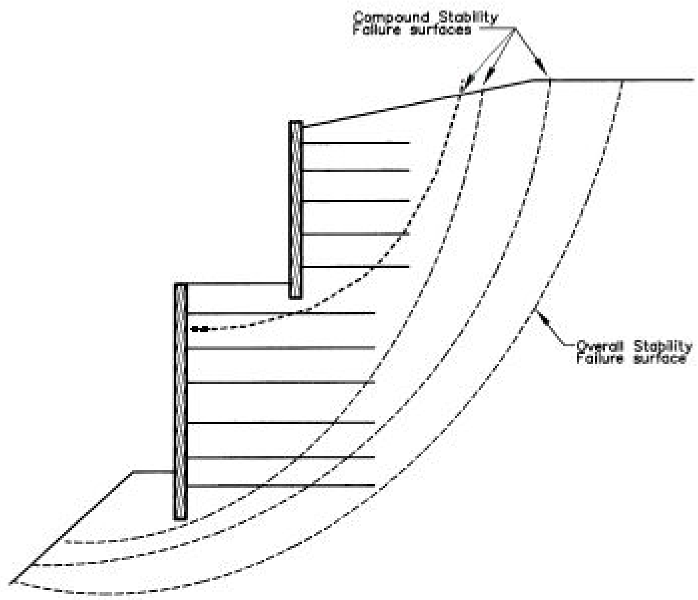


Figure 11.10.4.3-1 Overall and Compound Stability of Complex MSE Wall Systems.

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.

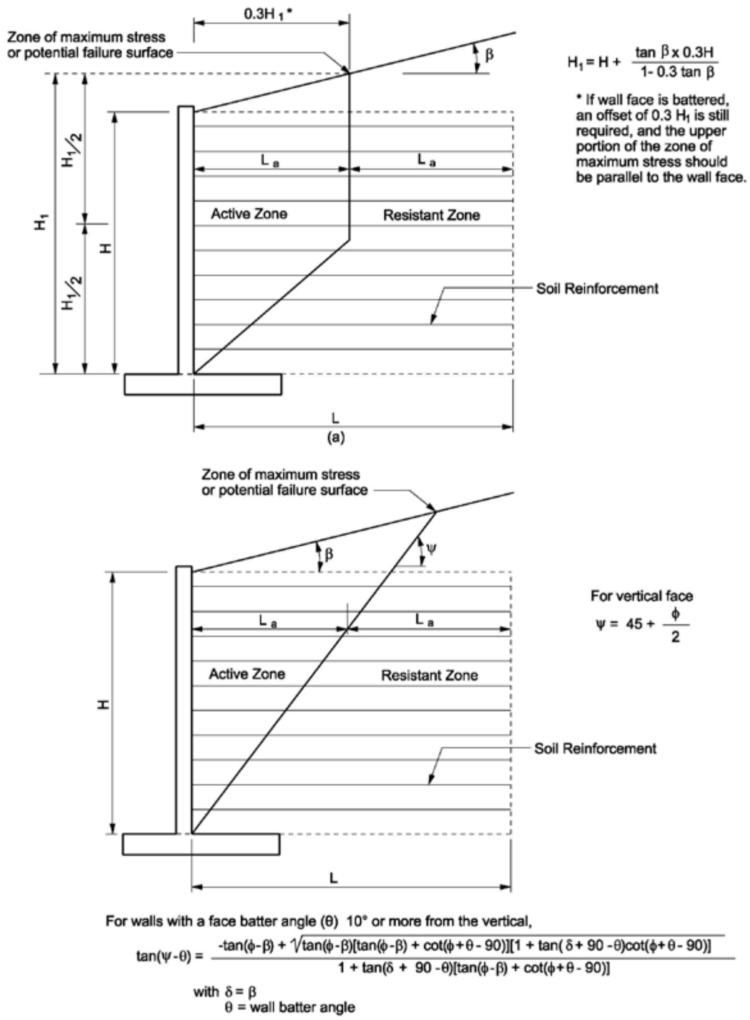


Figure 14.6-5

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

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

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.

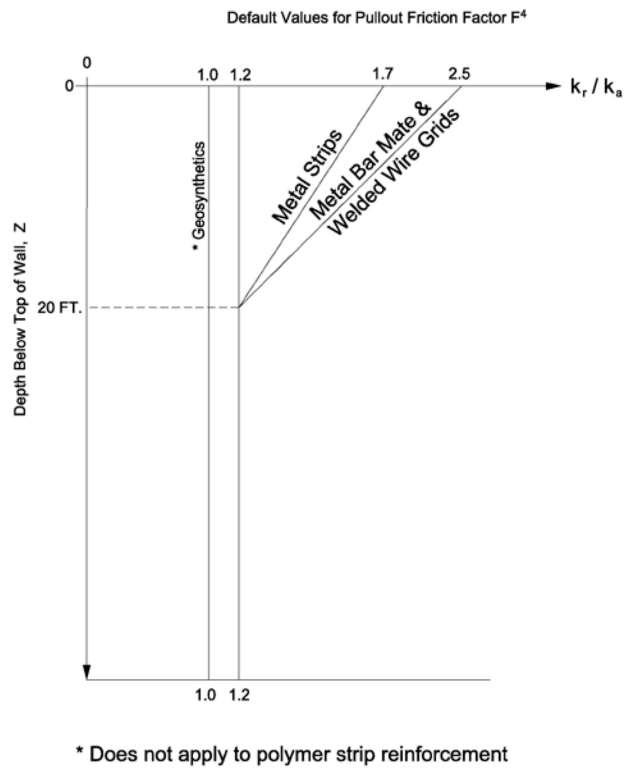


Figure 14.6-6
Variation of the Coefficient of Lateral Stress Ratio with Depth
(Source *AASHTO LRFD*)

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 σ_V. For the simplified method, vertical stress for the maximum reinforcement load calculations are shown in Figure 14.6-7 .

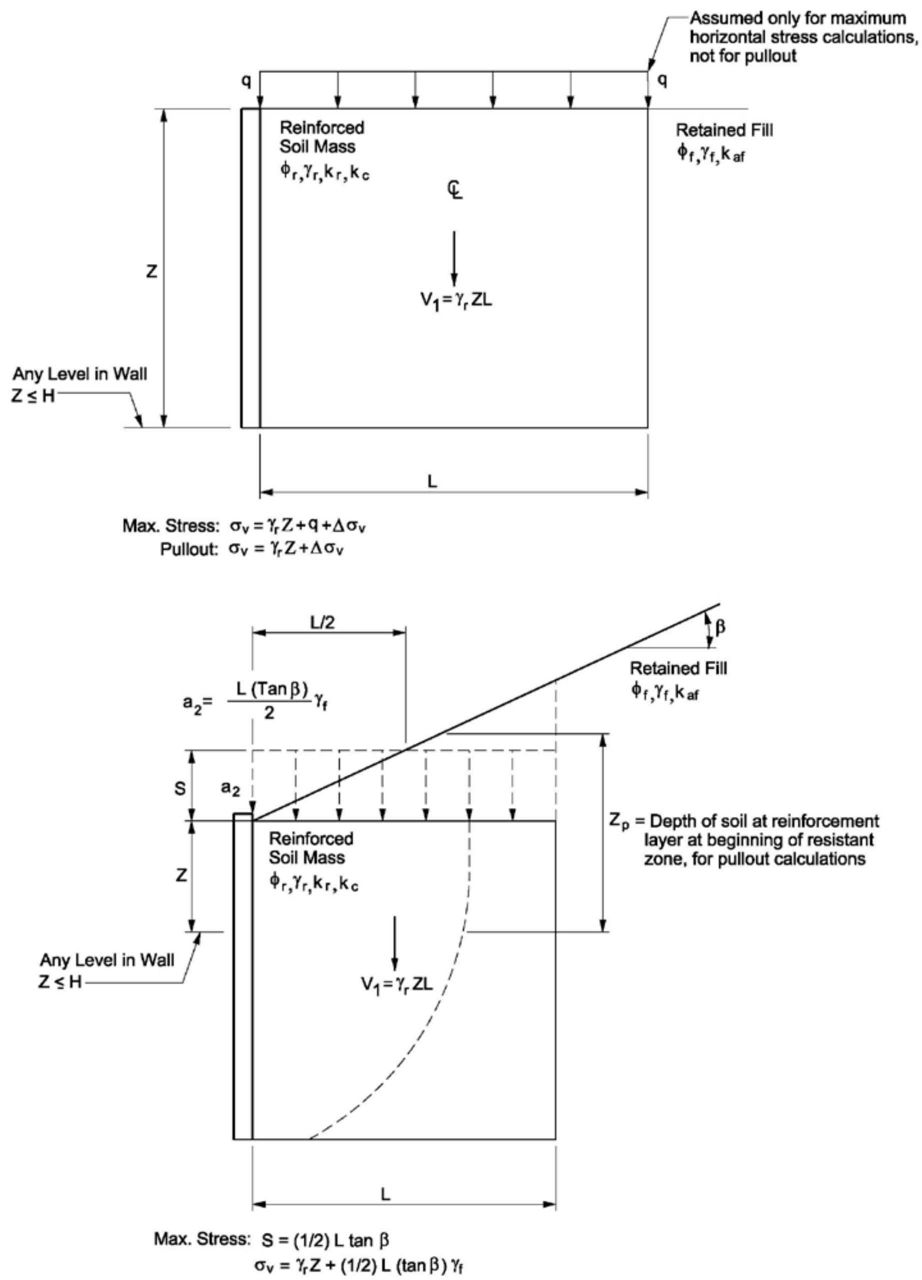


Figure 14.6-7

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 [11.10.6.4.1]**

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

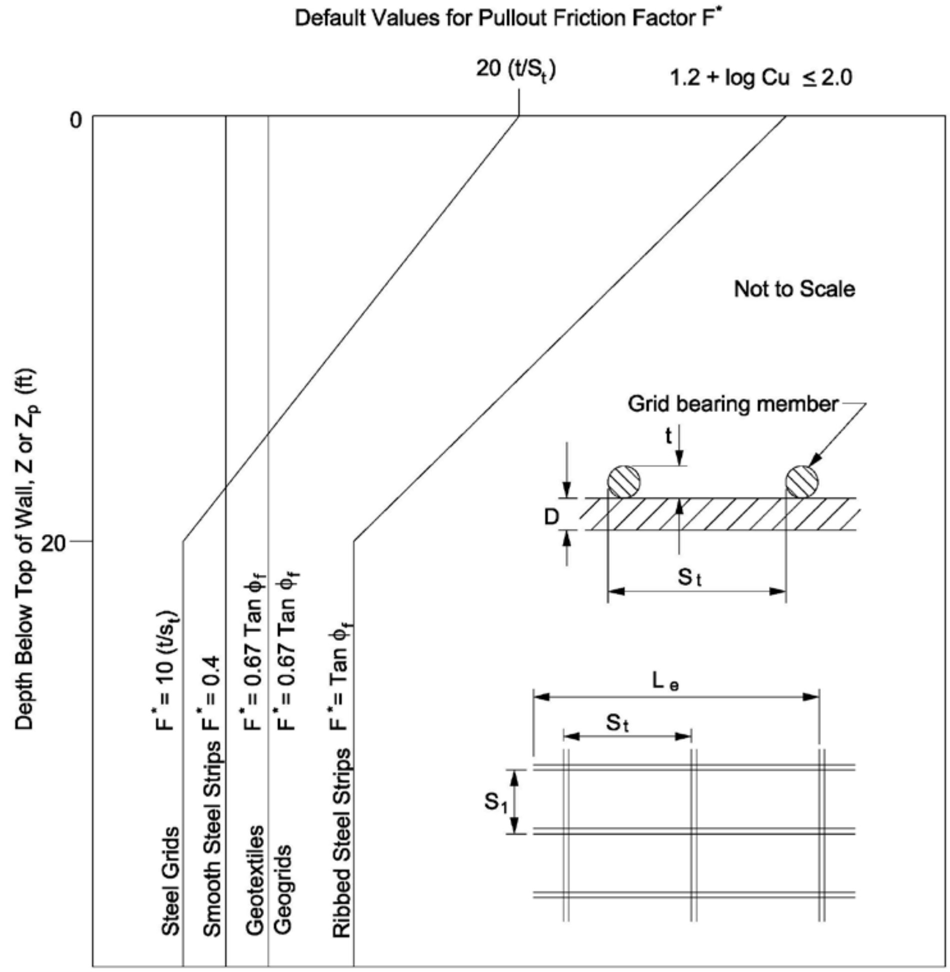


Figure 14.6-8
Default Values of F*
(Source: **LRFD [Figure 11.10.6.3.2-2]**)

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 Grid Reinforcement • Static Loading	 0.75 0.65
Geosynthetic reinforcement	• Static Loading	0.90

Table 14.6-3
Resistance Factor for Tensile and Pullout Resistance
(Source **LRFD [Table 11.5.7-1]**)

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 [11.10.6.4.2]**. 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]**.

Steel Reinforcement

Capacity of the connection shall be tested per **LRFD [5.10.8.3].** 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 [6.13]**. Connection material shall also be designed to accommodate loss due to corrosion.

Geosynthetic Reinforcement

The portion of the connection embedded in the concrete facing shall be designed in accordance with **LRFD [5.10.8.3]**. 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 salt spray 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°, or as determined from Geotechnical analyses (maximum allowed is 36°)

5. Retained Soil

- Unit weight = 120 lb/ft^3
- Angle of internal friction = 30°, 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

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 11.5.7-1]**)

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, $$\sigma_v = \frac{\sum V_{tot}}{(B-2e)}$$

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 **LRFD [Equation 10.6.3.1.2a-1]**
$\sum V$	=	Summation of Vertical loads
B	=	Base width
e	=	Eccentricity
ϕ_b	=	0.55 **LRFD [Table 11.5.7-1]**

14.7.1.2.4 Eccentricity Check

The eccentricity check shall be performed in accordance with **LRFD [11.6.3.3].** The location of the resultant force should be within the middle two-thirds of the base width (e<B/3) for footings on soil, and within nine-tenths of the base (e<0.45B) 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 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.

14.7.1.4 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 Engineering 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 [Table 11.5.7-1]**

9. Bearing Resistance Factors

   ϕ_{b} = 0.55 **LRFD [Table 11.5.7-1]**

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 standard height of these baskets may vary from 1 foot 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 =
    - o Use value provided by Geotechnical Engineer
- Rock-infill unit weight =
    - o 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 grouted in place and 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, sub-drainage 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-term 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.

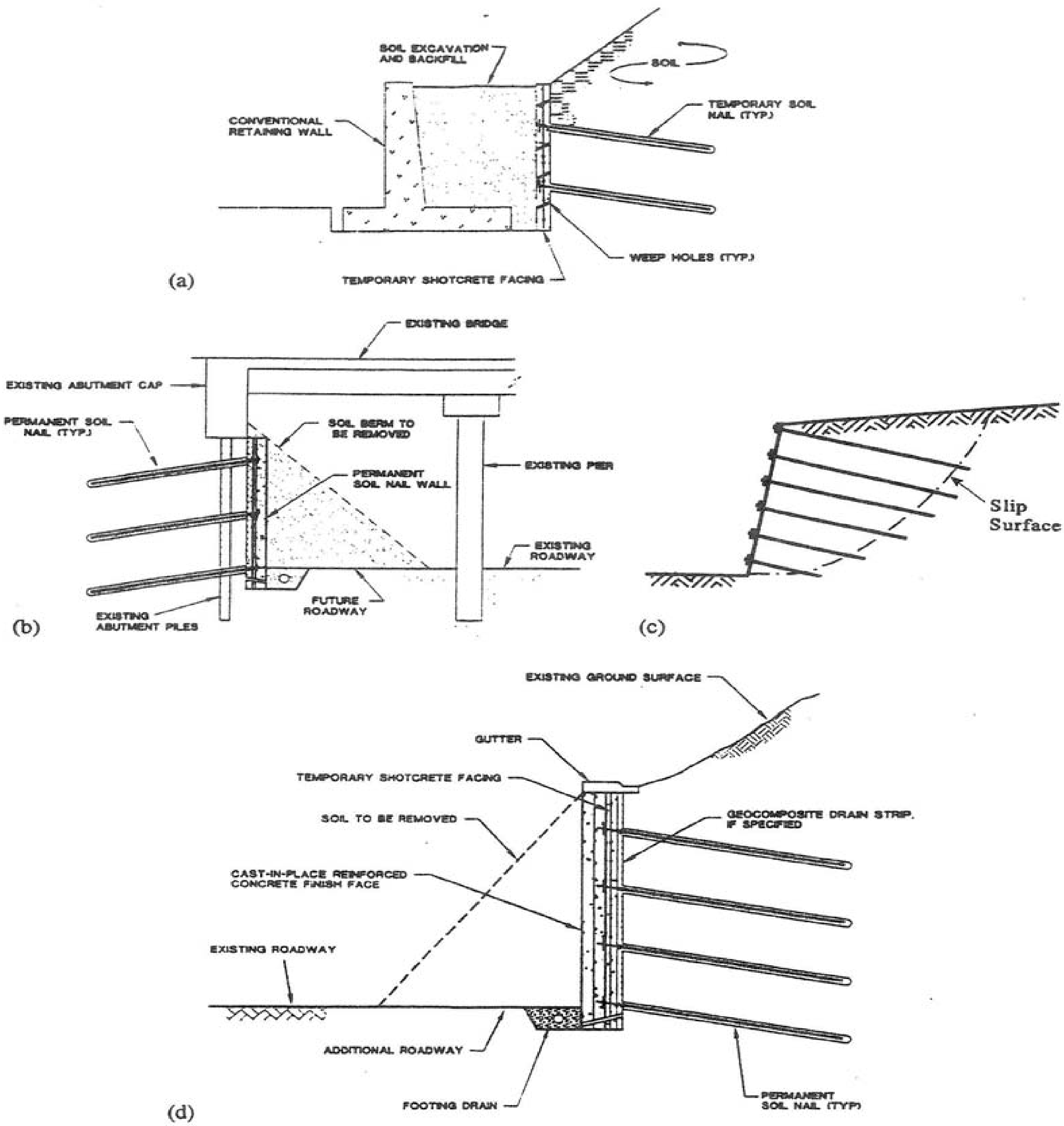


Figure 14.9-1
In-Situ Soil Nailed Walls
(Source: Earth Retaining Structures, 2008)

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, various sizes, 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 [3.11.5.6]** 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 [11.9]**. 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.

All areas of permanent exposed steel sheet piling above the ground line shall be coated or painted prior to driving. 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.

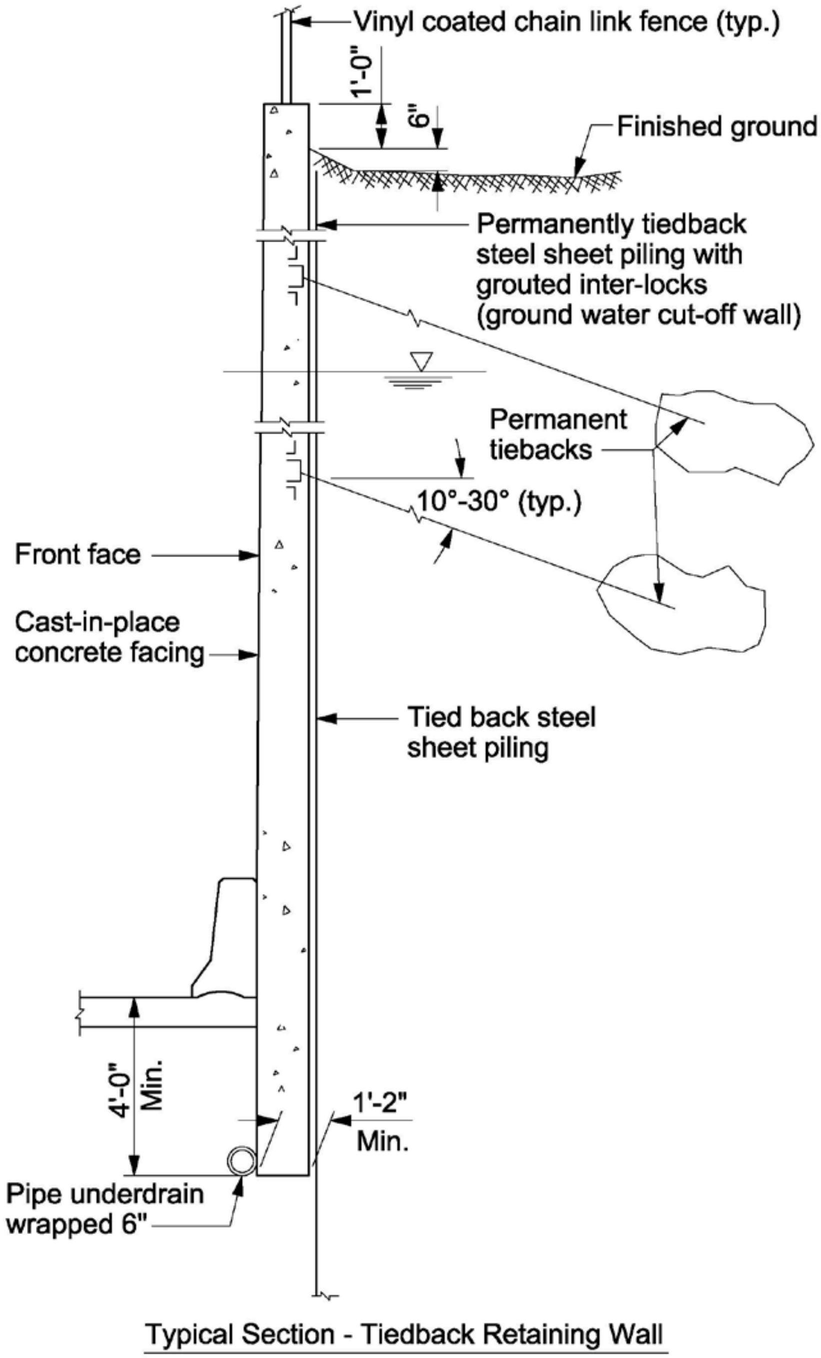


Figure 14.10-1
Typical Anchored Sheet Pile Wall

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 [Table 11.5.7-1]**
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 Soldier Pile Walls

Soldier pile walls are comprised of discrete vertical elements (usually steel H piles) and facing members (temporary and/or permanent) that extend between the vertical elements.

14.11.1 Design Procedure for Soldier Pile Walls

LRFD [11.8] Non-Gravity Cantilevered Walls covers the design of soldier pile walls. A simplified earth pressure distribution diagram is shown in **LRFD [3.11.5.6]** for permanent soldier pile 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, soldier pile walls have discrete vertical wall elements. Using "Broms" method for designing drilled shafts is also acceptable.

The maximum spacing between vertical supporting elements (piles) depends on the wall height and the design parameters of the foundation soil. Spacing of 6 to 12 feet is typical. The piles are set in drilled holes and concrete is placed in the hole after the post is set. The pile 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 structural thickness on wall facings shall be 6 inches for precast panels and 10 inches with cast-in-place concrete.

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 facings 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 soldier pile 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 soldier pile 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 (unit weight, angle of internal friction, and cohesion). Both drained and undrained parameters shall be considered.

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

   Use values provided by the Geotechnical Engineer (unit weight, angle of internal friction, and cohesion). Both drained and undrained parameters shall be considered.

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 soldier pile 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.

18. Location of name plate.

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
- Soldier Pile Walls
- Steel Sheet Piling Walls

14.15.2 Special Provisions

The Bureau of Structures has Special Provisions for:

- Wall Modular Block Gravity Landscape, Item SPV.0165.
- Wall Modular Block Gravity, Item SPV.0165.
- Wall Modular Block Mechanically Stabilized Earth, Item SPV.0165
- Wall Concrete Panel Mechanically Stabilized Earth, Item SPV.0165
- Wall Wire Faced Mechanically Stabilized Earth, Item SPV.0165. and Presstressed Precast Concrete Panel, Item SPV.0165
- Geosynthetic Reinforced Soil Abutment, Item SPV.0165
- Temporary Wall Wire Faced Mechanically Stabilized Earth, Item SPV.0165

- *Wall Gabion**
- *Wall Modular Bin or Crib**
- *Wall CIP Facing Mechanically Stabilized Earth**

** SPV under development. Contact the Bureau of Structures for usage.*

Note that the use of QMP Special Provisions began with the December 2014 letting or prior to December 2014 letting at the Region’s request. Special provisions are available on the Wisconsin Bridge Manual website.

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 Bureau of Structures 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 14.16.3.

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 (recommend a minimum of 15 weeks) but, to be considered for a particular WisDOT project, must be approved prior to the bid opening date. The material should be clearly detailed and presented according to the prescribed outline.

After final review and approval of comments with the Bureau of Structures, 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.

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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".

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

Table 17.2-7

Commonly Used Live Load Distribution Factors for Girder Structures

WisDOT exception to AASHTO:

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

WisDOT exception to AASHTO:

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

WisDOT policy item:

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

The following variables are used in Table 17.2-7:

S = Spacing of beams (feet)

L = Span length (feet)

t_s = Depth of concrete slab (inches)

K_g = Longitudinal stiffness parameter (inches^4)

N_b = Number of beams or girders

g = Distribution factor

e = Correction factor for distribution

d_e = Distance from the exterior web of exterior beam to the interior edge of curb or traffic barrier (feet)

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

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

Where:

L = Span length (feet)

t_s = Depth of concrete slab (inches)

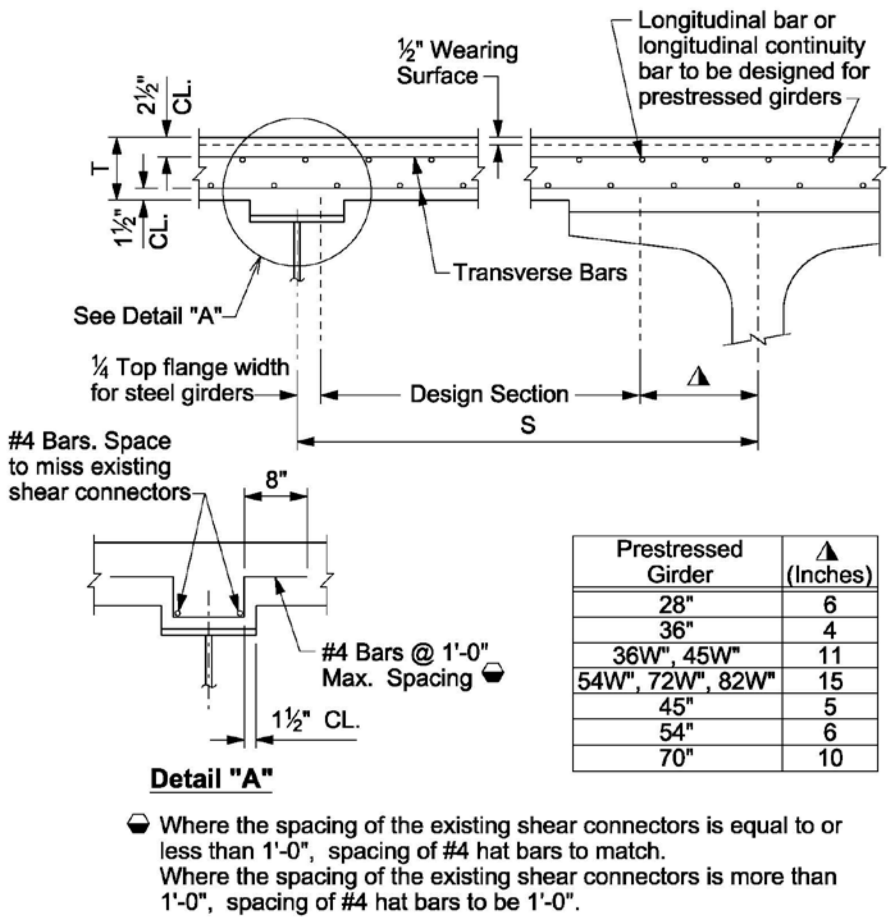
K_g = Longitudinal stiffness parameter (inches^4)

θ = Skew angle (degrees)

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

from the centerline of support. For prestressed concrete girders, this distance is equal to the values presented in Figure 17.5-1, along with bar locations and clearances.

Note: Transverse reinforcing steel requirements (bar size and spacing) are determined for both positive moment requirements and negative moment requirements, and the same reinforcing steel is used in both the top and bottom of slab as shown in Table 17.5-1 and Table 17.5-2. Longitudinal reinforcement in Table 17.5-3 and Table 17.5-4 is based on a percentage of the bottom transverse reinforcement required *by actual design calculations* (not a percentage of what is in the tables). **The tables should be used for deck reinforcement, with continuity bars in prestressed girder bridges being the only deck reinforcement requiring calculation.**



Prestressed Girder	△ (Inches)
28"	6
36"	4
36W", 45W"	11
54W", 72W", 82W"	15
45"	5
54"	6
70"	10

Figure 17.5-1
Transverse Section thru Slab on Girders

For skews of 20° and under, place transverse bars along the skew. For skews greater than 20°, place transverse bars perpendicular to the girders.

Detail "A", as presented in Figure 17.5-1, should be used for decks when shear connectors extend less than 2 inches into the slab on steel girder bridges or 3 inches on prestressed concrete girder bridges.

Several transverse reinforcing steel tables are provided in this chapter. The reinforcing steel in Table 17.5-1 and Table 17.5-2 does not account for deck overhangs. However, the minimum amount of reinforcing steel required in the deck overhangs is presented in various design tables in 17.6.

The reinforcement shown in Table 17.5-1 and Table 17.5-2 is based on both the Strength I requirement and crack control requirement.

Crack control was checked in accordance with **LRFD [5.6.7]**. The bar spacing cannot exceed the value from the following formula:

$$s \leq \frac{700(\gamma)}{\beta_s f_s} - 2 d_c$$

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) $\leq 0.6 f_y$
d_c	=	Top concrete cover less ½ inch wearing surface plus ½ bar diameter or bottom concrete cover plus ½ bar diameter (inches)
h	=	Slab depth minus ½ inch wearing surface (inches)

WisDOT policy item:

The thickness of the sacrificial ½-inch wearing surface shall not be included in the calculation of d_c.

Table 17.5-1 and Table 17.5-2 were developed for specified values of the distance from the centerline of girder to the design section for negative moment. Those specified values – 0, 3, 6, 9, 12 and 18 inches – were selected to match values used in **AASHTO [Table A4-1]**. For a girder in which the distance from the centerline of girder to the design section for negative 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 [5.10.6]** as follows:

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

and

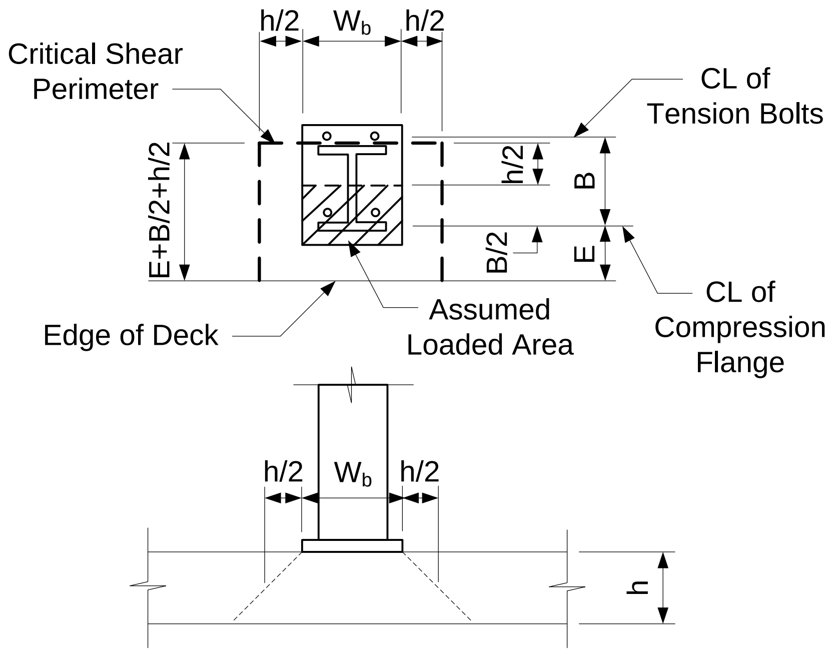


Figure 17.6-6
Assumed Load Distribution for Punching Shear

As used in Figure 17.6-6:

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

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

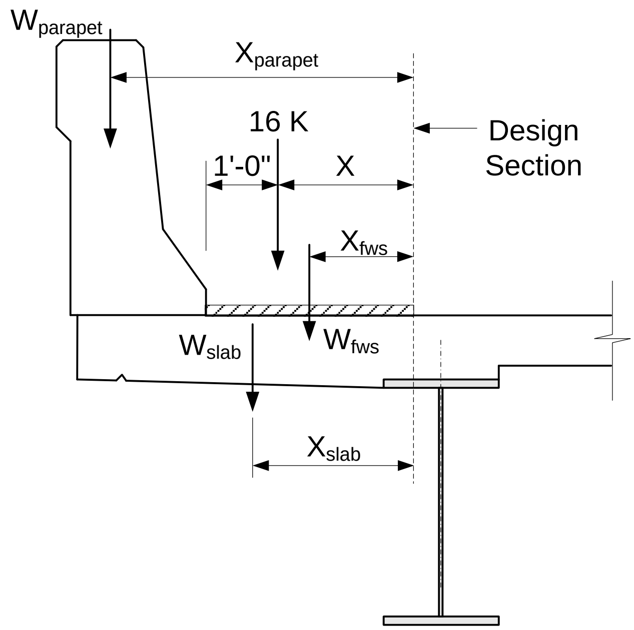


Figure 17.6-7
Design Case 3

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

Equivalent strip = 45.0 + 10.0X

Where:

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

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

Based on the computations for the three design cases, the controlling design case and design location are identified. The factored design moment is used to compute the required reinforcing steel. Cracking in the overhang must be checked for the service limit state in accordance with **LRFD [5.6.7]**. The controlling overhang reinforcement for cantilever deck slabs is shown in Table 17.6-2 and Table 17.6-3 for single slope and sloped face concrete parapets, and in Table 17.6-4 and Table 17.6-5 for steel railing Type “NY”/“M”. Type “W” railing is no longer allowed on girder structures.

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

WisDOT exception to AASHTO:

A 3-inch minimum panel thickness is used, even though **LRFD [9.7.4.3.1]** specifies a minimum thickness of 3.5 inches.

The decision to use a 3-inch minimum panel was based on the successful use of 3-inch panels by other agencies over many years. In addition, a minimum of 5 inches of cast-in-place concrete is preferred for crack control and reinforcing steel placement. A 3.5-inch panel thickness would require an 8.5-inch deck, which would not allow direct substitution of panels for a traditionally designed 8-inch deck.

A study performed at Iowa State University determined that a 3-inch thick panel with coated 3/8-inch strands at midthickness spaced at 6 inches, along with epoxy-coated 6 x 6 – D6 x D6 welded wire fabric, was adequate to prevent concrete splitting during strand detensioning. The use of #3 bars placed perpendicular to the strands at 9" spacing also prevents concrete splitting.

Panel thicknesses were increased by ½ inch whenever a strand spacing of less than 6 inches was required. Strands with a ½-inch diameter were used in panels 3½ inches thick or greater when 3/8-inch strands spaced at 6 inches were not sufficient.

The allowable tensile stress in the panels, as presented in **LRFD [Table 5.9.2.3.2b-1]**, is as follows:

$0.0948\lambda\sqrt{f'_c} \leq$ 0.3 ksi ; where λ = conc. density modification factor **LRFD [5.4.2.8]**, and has a value of 1.0 for normal weight conc.

This allowable tensile stress limit is based on f'_c in units of ksi and is for components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions.

The transfer length of the strands is assumed to be 60 strand diameters at a stress of 202.5 ksi. The development length, L_d, of the strands, as presented in **LRFD [5.9.4.3.2]**, is assumed to be as follows:

$$L_d = k\left(f_{ps} - \frac{2}{3}f_{pe}\right)d_b$$

Where:

k	=	1.0 for pretensioned members with a depth less than 24 inches
d_b	=	Nominal strand diameter (inches)
f_{ps}	=	Average stress in prestressing steel at the time when the nominal resistance of the member is required (ksi)

f_{pe} = Effective stress in prestressing steel after losses (ksi)

L_d = Development length beyond critical section (inches)

The minimum panel width is the length required for the panel to extend 4" onto the top flange as shown in Table 17.10-1. A linear reduction in f_{pe} is required if the panel width is less than two times the development length. The values shown in Table 17.10-1 consider this linear reduction.

The designs in Table 17.10-1 are based on uncoated prestressing strands. Grit-impregnated, epoxy-coated strands cost four times as much as uncoated strands but require about half the transfer and development length as uncoated strands. A cover of 1 1/4 inches is adequate to provide protection from chlorides for uncoated strands using a 5 ksi concrete mix. However, for bridges with high traffic volume, a 6 ksi mix is recommended.

LRFD [9.7.4.3.2] specifies that the strands need not extend beyond the panels into the cast-in-place concrete above the beams. This simplifies construction of the panels at the plant since they can be saw cut to the required length. Installation in the field is also simplified because extended strands may interfere with girder shear connectors. As a substitute for the strands that don't extend out of the panels, #4 bars spaced at twice the spacing of the transverse bars are placed on top of the panels over the girders in the cast-in-place concrete. These bars anchor the panels together to prevent or reduce longitudinal cracking over the ends of the panels and also resist any positive continuity moments that may develop. Also by not extending the strands into the cast-in-place concrete, the uncoated strands are not exposed to chlorides that may seep through cracks that may develop in the cast-in-place concrete.

LRFD [5.6.3.3] requires that the moment capacity of a flexural member be greater than the cracking moment based on the modulus of rupture. This requirement may be waived if the moment capacity is greater than 1.33 times the factored design moment. The purpose of this requirement is to provide a minimum amount of reinforcement in a flexural member so that a flexural failure will not be sudden or occur without warning. Tests have shown that for slabs on girders, the failure mode is a punching shear failure and not a flexural failure. ACI 10.5.4 also recognizes the difference between slabs and beams and does not require the same minimum reinforcement for slabs. For these reasons, **LRFD [5.6.3.3]** was not considered in the designs of the panels shown in Table 17.10-1. However, panels with a width of 6 feet or more meet the requirements of **LRFD [5.6.3.3]**.

17.10.3 Transverse Reinforcement for Cast-in-Place Concrete on Deck Panels

The design of the transverse reinforcing steel in the cast-in-place concrete placed on deck panels is based on *AASHTO LRFD*. The live load moments used to determine the size and spacing of the transverse reinforcing bars placed in the top of the cast-in-place concrete are from **LRFD [Table A4-1]**. The reinforcing steel in the cast-in-place concrete is also designed for a future wearing surface of 20 psf. With stay-in-place forms, there are no negative moments from the dead load of the cast-in-place concrete. The required reinforcing steel shown in Table 17.10-2 is based on both the strength requirement and crack control requirement.

Crack control was checked in accordance with **LRFD [5.6.7]** and as shown in 17.5.3.1. A concrete strength of 4 ksi was assumed, and the haunch height over the girders was not considered.

The distance from the centerline of the girder to the design section is from **LRFD [4.6.2.1.6]**. For prestressed concrete girders, use the values in Figure 17.5-1.

The reinforcing steel in Table 17.10-2 does not account for deck overhangs. However, Table 17.6-2, Table 17.6-3, Table 17.6-4 and Table 17.6-5 provide the minimum reinforcing steel required in the overhangs. Also for any portion of a deck not supported by deck panels, use Table 17.5-1 for determining the required reinforcing steel.

17.10.3.1 Longitudinal Reinforcement

For continuous prestressed concrete girders, the longitudinal reinforcing steel over the piers is the same as that required for a conventional deck. For steel girders, see 17.5.3.2 for longitudinal continuity reinforcement.

17.10.4 Details

Precast deck panels should extend a minimum of 1.5 inches beyond the face of concrete diaphragms at the substructure units. The transverse joints between panels in adjacent bays should be staggered, preferably a distance about 1/2 panel length. Staggering the joints helps to minimize transverse reflective cracking.

Panels should never rest directly on a girder flange. According to **LRFD [9.7.4.3.4]**, "The ends of the formwork panels shall be supported on a continuous mortar bed or shall be supported during construction in such a manner that the cast-in-place concrete flows into the space between the panel and the supporting component to form a concrete bedding." The minimum width of bearing on the flange of a girder for both concrete and mortar or grout support is 3 inches. See Figure 17.10-1 and Figure 17.10-2 for additional information.

High-density expanded polystyrene is used to support the panels prior to the placement of the cast-in-place concrete under the panel. The polystyrene is cut to the required haunch height so a constant slab thickness is maintained. High-density expanded polystyrene is available in different strengths, and it is the responsibility of the contractor to determine the strength required based on the vertical load that must be resisted. Fiber board or sheathing panel supports are not allowed because the slight deflection of polystyrene compresses the concrete underneath the panel and results in less reflective longitudinal cracking along the panel edge.

When panels are supported on grout, the main function of the polystyrene is to form the haunch height and to form a dam for the grout placement. The grout must be placed immediately before placement of the panels. It is important that enough grout be placed so that the vertical load from the panels is supported by the grout and not by the polystyrene.

Some agencies specify a maximum haunch height. When it is exceeded, they allow the contractor to thicken the slab. Wisconsin does not specify a maximum haunch height and leaves that decision to the designer, who is better informed to make that decision based on the specific situation of their project.

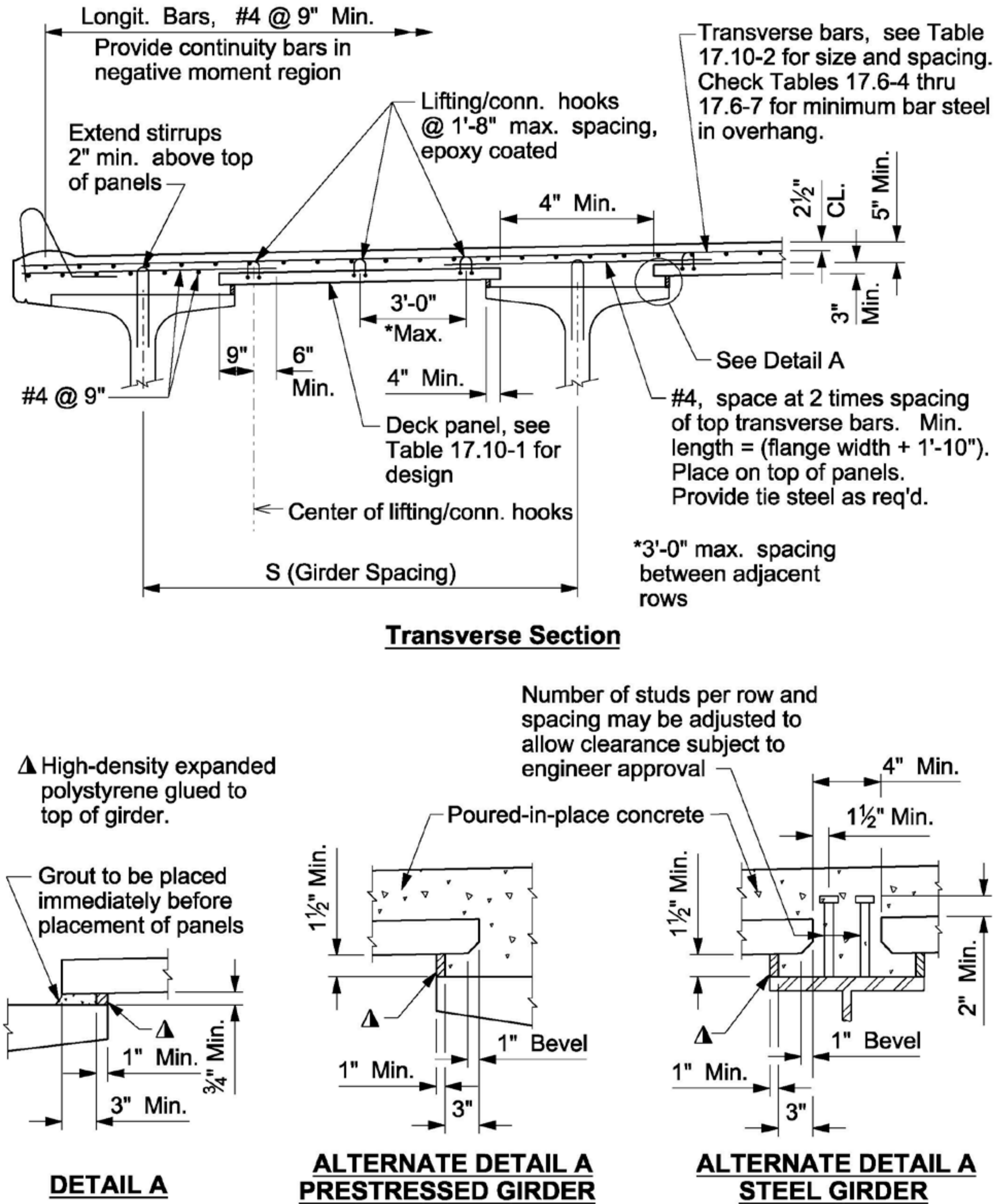
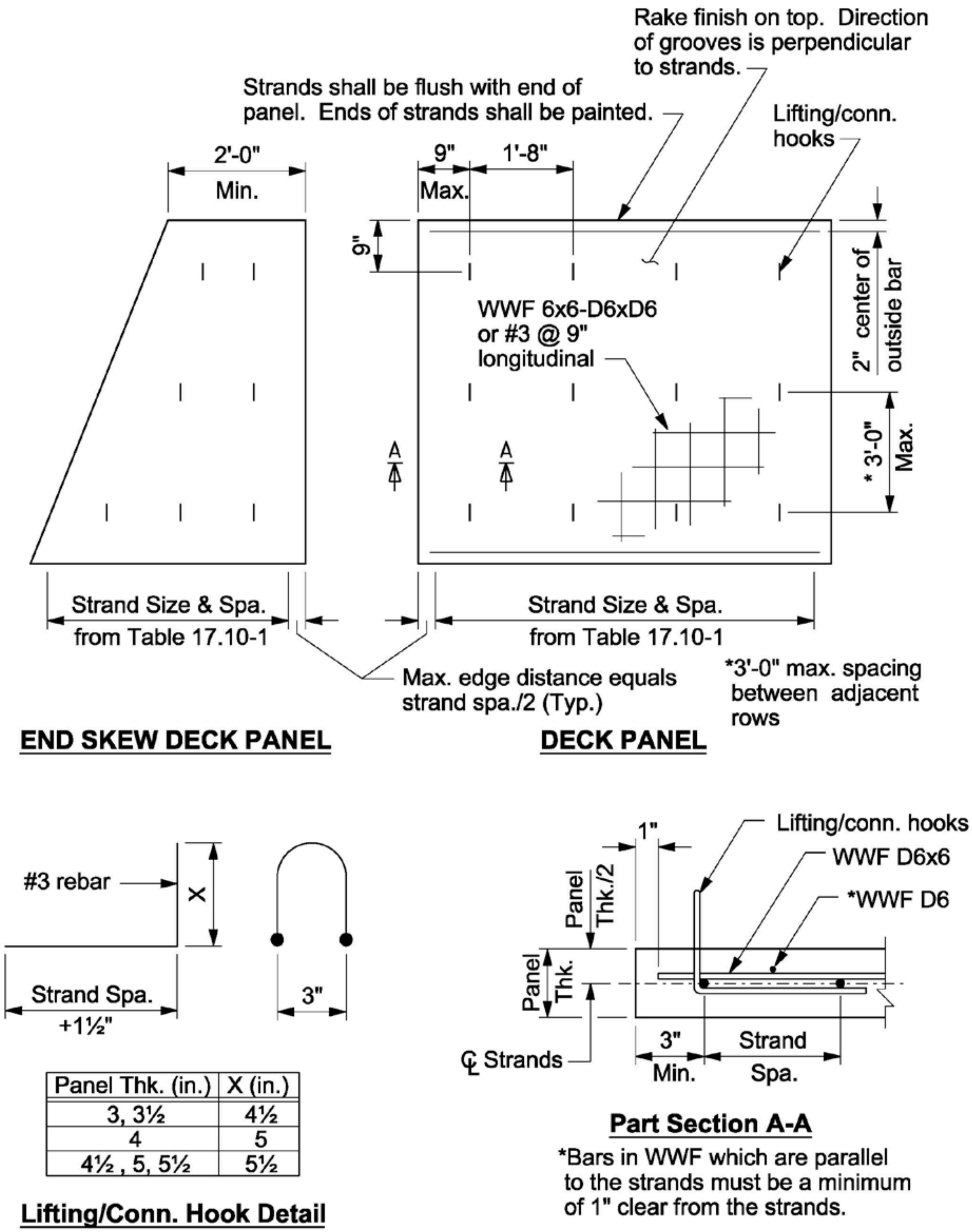


Figure 17.10-1
Transverse Section through Slab on Girders with Deck Panel and Details



Panel Thk. (in.)	X (in.)
3, 3½	4½
4	5
4½ , 5, 5½	5½

Figure 17.10-2
Deck Panel Details

Girder Spacing "S"	Panel Thick. (Inches)	Total Slab Thick. (Inches)	Top Flange Width (Inches)												
			12		16		18		24		30		48		
			Strand		Strand		Strand		Strand		Strand		Strand		
			Spa. Inches	Pi Kips	Spa. Inches	Pi Kips	Spa. Inches	Pi Kips	Spa. Inches	Pi Kips	Spa. Inches	Pi Kips	Spa. Inches	Pi Kips	
4'-6"	3	8	10	13.17	10	12.33	10	11.92	10	11.08	10	11.08	10	11.08	3/8" Diameter Strands
4'-9"	3	8	10	13.58	10	12.75	10	12.75	10	11.50	10	11.08	10	11.08	
5'-0"	3	8	10	14.42	10	13.58	10	13.17	10	12.33	10	11.08	10	11.08	
5'-3"	3	8	10	14.83	10	14.00	10	13.58	10	12.75	10	11.92	10	11.08	
5'-6"	3	8	10	15.67	10	14.83	10	14.42	10	13.17	10	12.33	10	11.08	
5'-9"	3	8	10	16.50	10	15.67	10	15.25	10	14.00	10	13.17	10	11.50	
6'-0"	3	8	8	14.25	10	16.50	10	16.08	10	14.83	10	13.58	10	11.50	
6'-3"	3	8	8	15.45	8	14.25	10	16.92	10	15.67	10	14.42	10	11.92	
6'-6"	3	8	8	16.12	8	15.45	8	14.78	10	16.50	10	15.25	10	12.33	
6'-9"	3	8	8	17.12	8	16.12	8	15.78	8	14.25	10	16.08	10	13.17	
7'-0"	3	8	6	14.19	8	17.12	8	16.45	8	15.45	8	14.25	10	13.58	
7'-3"	3	8	6	14.94	6	14.19	6	13.62	8	16.12	8	15.12	10	14.42	
7'-6"	3	8	6	15.69	6	14.94	6	14.69	8	17.12	8	15.78	10	15.25	
7'-9"	3	8	6	16.44	6	15.69	6	15.44	6	14.44	8	16.78	10	16.50	
8'-0"	3	8	6	17.19	6	16.44	6	16.19	6	15.19	6	14.19	8	14.25	
8'-3"	3.5	8.5	6	16.76	6	16.01	6	15.76	6	14.76	6	13.47	8	14.14	
8'-6"	3.5	8.5	10	29.48	6	16.76	6	16.51	6	15.51	6	14.51	8	14.97	
8'-9"	3.5	8.5	8	26.44	10	30.06	10	29.06	6	16.26	6	15.26	8	15.97	
9'-0"	3.5	8.5	8	27.44	8	26.44	8	26.10	6	17.01	6	16.01	8	16.64	
9'-3"	3.5	8.5	8	28.77	8	27.77	8	27.10	10	30.06	6	16.76	6	14.01	
9'-6"	4	9	8	27.76	8	26.76	8	25.95	10	29.22	6	16.37	8	17.20	
9'-9"	4	9	8	29.09	8	27.76	8	27.43	10	30.62	6	17.12	6	14.37	
10'-0"	4	9	8	30.09	8	29.09	8	28.43	8	27.09	10	30.20	6	15.12	
10'-3"	4	9	6	25.48	8	30.09	8	29.76	8	28.09	8	26.76	6	15.87	
10'-6"	4	9	6	26.23	6	25.48	8	30.76	8	29.09	8	27.76	6	16.62	
10'-9"	4	9.5	6	26.73	6	25.73	6	25.23	8	29.43	8	27.76	6	16.12	
11'-0"	4	9.5	6	27.48	6	26.73	6	26.23	8	30.43	8	28.76	6	16.87	
11'-3"	4	9.5	6	28.48	6	27.48	6	26.98	6	25.73	8	30.09	10	30.20	1/2" Diameter Strands
11'-6"	4	9.5	6	29.48	6	28.48	6	27.98	6	26.73	6	25.23	8	25.95	
11'-9"	4	10	6	30.23	6	28.98	6	28.48	6	26.98	6	25.48	8	25.95	
12'-0"	4.5	10	6	29.62	6	28.62	6	28.12	6	26.62	6	25.37	8	26.50	
12'-3"	4.5	10	6	30.62	6	29.62	6	29.12	6	27.62	6	26.12	8	27.83	
12'-6"	5	10	6	30.34	6	29.34	6	28.84	6	27.59	6	26.34	8	28.28	
12'-9"	5	10.5	6	30.59	6	29.59	6	29.09	6	27.59	6	26.34	8	27.95	
13'-0"	5.5	10.5	6	30.36	6	29.36	6	29.11	6	27.61	6	26.36	8	28.77	
13'-3"	5.5	10.5	4	23.52	6	30.36	6	29.86	6	28.61	6	27.36	8	29.77	
13'-6"	5.5	10.5	4	24.18	4	23.52	4	23.18	6	29.36	6	28.11	8	30.77	

13'-9"	6	11	4	23.39	6	30.41	6	30.16	6	28.66	6	27.41	8	29.96	
14'-0"	6	11	4	24.06	4	23.39	4	23.06	6	29.66	6	28.16	8	30.96	

Table 17.10-1

Precast Prestressed Concrete Deck Panel Design Table

Notes:

- Designed per AASHTO LRFD Specifications with HL 93 Loading.
- f'c = 6.0 ksi
- f'ci = 4.4 ksi
- f'c slab = 4.0 ksi
- f's = 270 ksi (low relaxation)
- Design loading includes 20 psf for future wearing surface and 50 psf for construction load. Pi's in Table are a minimum and may be increased to a maximum of 0.75 x f_s x A_s. Strands are located at the centroid of the panels.

Girder Spacing "S"	Total Slab Thick. Inches	Distance From C/L of Girder to Design Section (Inches)					
		3	4	5	6	10	15
4'-6"	8	#4 @ 9	#4 @ 9.5	#4 @ 10	#4 @ 10	#4 @ 11.5	#4 @ 12.5
4'-9"	8	#4 @ 8	#4 @ 8.5	#4 @ 9	#4 @ 9.5	#4 @ 11	#4 @ 12.5
5'-0"	8	#4 @ 7	#4 @ 7.5	#4 @ 8	#4 @ 8.5	#4 @ 10.5	#4 @ 12.5
5'-3"	8	#4 @ 6.5	#4 @ 7	#4 @ 7.5	#4 @ 8	#4 @ 10	#4 @ 12
5'-6"	8	#4 @ 6.5	#4 @ 6.5	#4 @ 7	#4 @ 7.5	#4 @ 9.5	#4 @ 12
5'-9"	8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 7	#4 @ 9	#4 @ 11.5
6'-0"	8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 8.5	#4 @ 11
6'-3"	8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 8.5	#4 @ 11
6'-6"	8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 8	#4 @ 10.5
6'-9"	8	#4 @ 6	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 7.5	#4 @ 10.5
7'-0"	8	#5 @ 7.5	#5 @ 8	#4 @ 6.5	#4 @ 6.5	#4 @ 7	#4 @ 10
7'-3"	8	#5 @ 7.5	#5 @ 7.5	#4 @ 6	#4 @ 6.5	#4 @ 6.5	#4 @ 10
7'-6"	8	#5 @ 7.5	#5 @ 7.5	#4 @ 6	#4 @ 6.5	#4 @ 6.5	#4 @ 9.5
7'-9"	8	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#4 @ 6.5	#4 @ 6.5	#4 @ 8.5

8'-0"	8	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#4 @ 6	#4 @ 6.5	#4 @ 8
8'-3"	8.5	#5 @ 7.5	#5 @ 8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 8
8'-6"	8.5	#5 @ 7.5	#5 @ 8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 8
8'-9"	8.5	#5 @ 7.5	#5 @ 7.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 7.5
9'-0"	8.5	#5 @ 7.5	#5 @ 7.5	#5 @ 8	#4 @ 6.5	#4 @ 6.5	#4 @ 7.5
9'-3"	8.5	#5 @ 7.5	#5 @ 7.5	#5 @ 8	#4 @ 6.5	#4 @ 6.5	#4 @ 7
9'-6"	9	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8.5	#5 @ 9.5	#4 @ 7
9'-9"	9	#5 @ 7.5	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 9.5	#4 @ 7
10'-0"	9	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 8	#5 @ 9	#4 @ 6.5
10'-3"	9	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 8.5	#5 @ 9.5
10'-6"	9	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8.5	#5 @ 9
10'-9"	9.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 8.5	#5 @ 9
11'-0"	9.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 8.5	#5 @ 9
11'-3"	9.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 9
11'-6"	9.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 8	#5 @ 9
11'-9"	10	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 8	#5 @ 9
12'-0"	10	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 9
12'-3"	10	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 8	#5 @ 8.5
12'-6"	10	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8.5
12'-9"	10.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8.5
13'-0"	10.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 8	#5 @ 8.5
13'-3"	10.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8.5
13'-6"	10.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8.5
13'-9"	11	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 8	#5 @ 8
14'-0"	11	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8

Table 17.10-2
Transverse Reinforcing Steel for Deck Slabs on Precast Concrete Deck Panels

Notes:

- Designed per AASHTO LRFD with HL-93 Loading.
- f'c deck = 4.0 ksi
- fy = 60 ksi
- Steel is 2 ½" clear from top of slab. Designed for 20 psf future wearing surface. "Total Slab Thickness" includes thickness of deck panel and poured in place concrete.
- Overhang deck steel may require greater than the number 4 or 5 bar as indicated in 17.6.2.

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.

WisDOT policy item:

Slab bridges, due to camber required to address future creep deflection, do not ride ideally for the first few years of their service life and present potential issues due to ponding. As such, if practical (e.g. not excessive financial implications), consideration of other structure types should be given for higher volume/higher speed facilities, such as the Interstate. Understanding these issues, the Regions have the responsibility to make the final decision on structure type with respect to overall project cost, with BOS available for consultation.

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 [C5.4.2.4]**

= $33{,}000\ K_1\ w_c^{1.5}\ (f'_c)^{1/2} = 3800$ ksi

Where:

K_1 = 1.0

w_c = 0.150 kcf, unit weight of concrete

n = $E_s / E_c = 8$ **LRFD [5.6.1]** (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 structure

- Ignore any influence of ADTT on multiple presence factor, m, in **LRFD [Table 3.6.1.1.2-1]** that would reduce force effects, Q_i , for slab bridges.

- Ignore reduction factor, r, for skewed slab bridges in **LRFD [4.6.2.3]** that would reduce longitudinal force effects, Q_i .

18.3.3 Strength Limit State

Strength I Limit State shall be applied to ensure that strength and stability are provided to resist the significant load combinations that a bridge is expected to experience during its design life **LRFD [1.3.2.4]**. The total factored force effect, Q, must not exceed the factored resistance, R_r , as shown in the equation in 18.3.2.1.

Strength I Limit State **LRFD [3.4.1]** will be used for:

- Designing longitudinal slab reinforcement for flexure

- Designing transverse slab reinforcement over the piers for flexure

- Checking shear (two-way) in slab at the piers

- Checking uplift at the abutments

- Checking longitudinal slab reinforcement for tension from shear

18.3.3.1 Factored Loads

The value of the load modifier, η_i , is 1.00, as stated in 18.3.2.2.

Strength I Limit State will be used to design the structure for force effects, Q_i , due to applied dead loads, DC and DW (including future wearing surface), defined in 18.4.2 and appropriate (HL-93) live loads, LL and IM, defined in 18.4.3.1. When sidewalks are present, include force effects of pedestrian live load, PL, defined in 18.4.3.2.

The load factor, γ_i , is used to adjust force effects on a structural element. This factor accounts for variability of loads, lack of accuracy in analysis, and the probability of simultaneous occurrence of different loads.

For Strength I Limit State, the values of γ_i for each applied load, are found in **LRFD [Tables 3.4.1-1 and 3.4.1-2]** and their values are: γ_{DC} = 1.25/0.90, γ_{DW} = 1.50/0.65, γ_{LL+IM} = γ_{PL} = 1.75. The values for γ_{DC} and γ_{DW} have a maximum and minimum value.

Therefore, for Strength I Limit State:

$$Q = 1.0\ [\ 1.25(DC) + 1.50(DW) + 1.75((LL + IM) + PL)\ ]$$

Where DC, DW, LL, IM, and PL represent force effects due to these applied loads. The load factors shown for DC and DW are maximum values. Use maximum or minimum values as shown in **LRFD [Table 3.4.1-2]** to calculate the critical force effect.

18.3.3.2 Factored Resistance

The resistance factor, ϕ, is used to reduce the computed nominal resistance of a structural element. This factor accounts for variability of material properties, structural dimensions and workmanship, and uncertainty in prediction of resistance.

The resistance factors, ϕ, for Strength Limit State **LRFD [5.5.4.2]** are:

- ϕ = 0.90 for flexure & tension (for tension-controlled reinforced concrete sections as defined in **LRFD [5.6.2.1]**)
- ϕ = 0.90 for shear and torsion

The factored resistance, R_r (M_r, V_r, T_{cap}), associated with the list of items to be designed/checked using Strength I Limit State in 18.3.3, are described in the following sections.

18.3.3.2.1 Moment Capacity

Stress is assumed proportional to strain below the proportional limit on the stress-strain diagram. Tests have shown that at high levels of stress in concrete, stress is not proportional to strain. Recognizing this fact, strength analysis takes into account the nonlinearity of the stress-strain diagram. This is accomplished by using a rectangular stress block to relate the concrete compressive stress distribution to the concrete strain. The compressive stress block has a uniform value of $\alpha_1 \cdot f'_C$ over a zone bounded by the edges of the cross section and a straight line located parallel to the neutral axis at the distance a = $\beta_1 \cdot (c)$ from the extreme compression fiber. The distance (c) is measured perpendicular to the neutral axis. The factor α_1 shall be taken as 0.85 for concrete strengths not exceeding 10.0 ksi and the factor β_1 shall be taken as 0.85 for concrete strengths not exceeding 4.0 ksi **LRFD [5.6.2.2]**. Strength predictions using this method are in agreement with strength test results. The representation of these assumptions is shown in Figure 18.3-1.

The moment capacity (factored resistance) of concrete components shall be based on the conditions of equilibrium and strain compatibility, resistance factors as specified in **LRFD [5.5.4.2]** and the assumptions outlined in **LRFD [5.6.2]**.

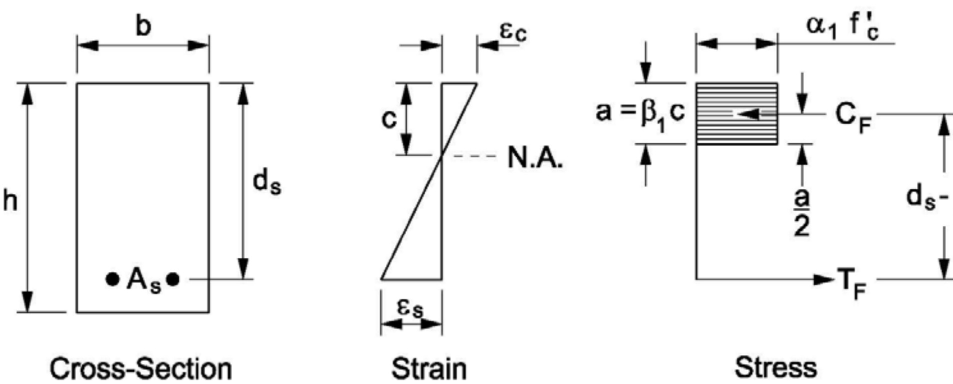


Figure 18.3-1
Stress / Strain on Cross - Section

Referring to Figure 18.3-1, the internal force equations are:

$C_F = \alpha_1 \cdot (f'_c)\ (b)\ (a) = 0.85\ (f'_c)\ (b)\ (a)$

$T_F = (A_s)\ (f_s)$

By equating C_F to T_F, and solving for the compressive stress block depth, (a), gives:

$a = A_s\ f_s\ /\ 0.85\ (f'_c)\ (b)$

Use ($f_s = f_y$) when the steel yields prior to crushing of the concrete. To check for yielding, assume ($f_s = f_y$) and calculate the value for (a). Then calculate the value for $c = a\ /\ \beta_1$ and d_s as shown in Figure 18.3-1. If $c\ /\ d_s$ does not exceed the value calculated below, then the reinforcement has yielded and the assumption is correct, as stated in **LRFD [5.6.2.1]**.

$c\ /\ d_s \leq 0.003\ /\ (0.003 + \varepsilon_{cl})$

ε_{cl} = compression controlled strain limit

for f_y = 60 ksi, ε_{cl} is 0.0020 per **LRFD [Table C5.6.2.1-1]**

if $c\ /\ d_s \leq 0.6$, then the reinforcement (f_y = 60 ksi) will yield and ($f_s = f_y$)

For rectangular sections, the nominal moment resistance, M_n , (tension reinforcement only) equals: **LRFD [5.6.3.2.3]**

$M_n = A_s\ f_s\ (d_s - a/2)$

The factored resistance, M_r , or moment capacity, shall be taken as: **LRFD [5.6.3.2.1]**

$M_r = \phi\ M_n = \phi\ A_s\ f_s\ (d_s - a/2)$

For tension-controlled reinforced concrete sections, the resistance factor, ϕ, is 0.90, therefore:

$$M_r = (0.9)\ A_s\ f_s\ (d_s - a/2)$$

18.3.3.2.2 Shear Capacity

The nominal shear resistance, V_n , for two-way action, shall be determined as: **LRFD [5.7.1.4, 5.12.8.6.3]**

$$V_n = (0.063 + 0.126 / \beta_c)\ \lambda\ (f'_c)^{1/2}\ b_o\ d_v \leq 0.126\ \lambda\ (f'_c)^{1/2}\ b_o\ d_v \quad \text{(kips)}$$

Where:

f'_c	=	4.0 ksi (for concrete slab bridges)
β_c	=	ratio of long side to short side of the rectangle through which the concentrated load or reaction force is transmitted
d_v	=	effective shear depth as determined in **LRFD [5.7.2.8]** (in)
b_o	=	perimeter of the critical section (in)
λ	=	conc. density modification factor ; for normal weight conc. = 1.0 , **LRFD [5.4.2.8]**

The factored resistance, V_r , or shear capacity, shall be taken as: **LRFD [5.7.2.1]**

$$V_r = \phi\ V_n$$

The resistance factor, ϕ, is 0.90, therefore:

$$V_r = (0.9)\ V_n$$

18.3.3.2.3 Uplift Check

The check of uplift at abutments does not use a factored resistance, but compares factored dead load and live load reactions.

18.3.3.2.4 Tensile Capacity – Longitudinal Reinforcement

The nominal tensile resistance, T_{nom} , for an area, A_s , of developed reinforcement, equals:

$$T_{nom} = A_s\ f_y$$

The factored resistance, T_{cap} , or tensile capacity, shall be taken as:

$$T_{cap} = \phi\ T_{nom} = \phi\ A_s\ f_y$$

For tension-controlled reinforced concrete sections, the resistance factor, ϕ, is 0.90, therefore:

$R_r = (1.0)\ R_n$ = (maximum allowable camber) / 3

18.3.5 Fatigue Limit State

Fatigue I Limit State shall be applied as a restriction on stress range as a result of a single design truck occurring at the number of expected stress range cycles **LRFD [1.3.2.3]**. The Fatigue I Limit State is intended to limit crack growth under repetitive loads to prevent fracture of the reinforcement during the design life of the bridge. The factored force effect (stress range), Q, must not exceed the factored resistance, R_r , as shown in the equation in 18.3.2.1.

For fatigue considerations, concrete members shall satisfy: **LRFD [5.5.3.1]**

$$\eta_i\ \gamma_i\ (\Delta f) \leq (\Delta F)_{TH}$$

Where:

γ_i	=	Load factor for Fatigue I Limit State
Δf	=	Force effect, live load stress range due to the passage of the fatigue truck (ksi)
$(\Delta F)_{TH}$	=	Constant-amplitude fatigue threshold (ksi)

Fatigue I Limit State **LRFD [3.4.1]** will be used for:

- Checking longitudinal slab reinforcement for fatigue stress range criteria

18.3.5.1 Factored Loads (Stress Range)

The value of the load modifier, η_i , is 1.00, as stated in 18.3.2.2.

Fatigue I Limit State will be used to analyze the structure for force effects, $Q_i = (\Delta f)$, due to applied (Fatigue Truck) live load, LL and IM, defined in 18.4.3.1.

For Fatigue I Limit State, the value of γ_i for the applied live load, is found in **LRFD [Table 3.4.1-1]** and its value is $\gamma_{LL+IM} = 1.75$.

Therefore, for Fatigue I Limit State:

$$Q = 1.0\ [\ 1.75(LL + IM)\ ]$$

Where LL and IM represent force effects, Δf, due to these applied loads.

18.3.5.2 Factored Resistance

The resistance factor, ϕ, for Fatigue Limit State, is found in **LRFD [C1.3.2.1]** and its value is 1.00 .

18.3.5.2.1 Fatigue Stress Range

The nominal resistance, $R_n = (\Delta F)_{TH}$, for fatigue stress range (for straight reinforcement), is: **LRFD [5.5.3.2]**

$$R_n = (\Delta F)_{TH} = 26 - 22 f_{min} / f_y \quad \text{(ksi)}$$

Where:

f_{min} = the minimum stress resulting from the factored Fatigue Truck live load, combined with the stress from the dead loads on the structure; positive if tension, negative if compression (ksi)

f_y = minimum yield strength (ksi), not to be taken less than 60 ksi nor greater than 100 ksi

The factored resistance, R_r (for f_y = 60 ksi), is:

$$R_r = \phi R_n = \phi (26 - 0.37 f_{min})$$

The resistance factor, ϕ, is 1.00, therefore:

$$R_r = (1.0) R_n = 26 - 0.37 f_{min} \quad \text{(ksi)}$$

use Design Lane Load portion of LL#2 for Live Load Case 1 and use Design Truck (+IM) portion of LL#2 for Live Load Case 2, as shown in Figure 17.2-11.

[2] (LL#3) is used to calculate negative live load moments between points of contraflexure and also reactions at interior supports. The points of contraflexure are located by placing a uniform load across the entire structure. For these moments and reactions, the results calculated from (LL#3) are compared with (LL#1) and (LL#2) results, and the critical value is selected.

[3] Used for design of interior strip only.

18.4.3.2 Pedestrian Live Load (PL)

For bridges designed for both vehicular and pedestrian live load, a pedestrian live load, PL, of 75 psf is used. However, for bridges designed exclusively for pedestrian and/or bicycle traffic, see *AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges* for live load. The dynamic load allowance, IM, is not applied to pedestrian live loads **LRFD [3.6.2]**.

Pedestrian loads are not applied to an interior strip for its design. For the design of exterior strips (edge beams), any pedestrian loads that are located directly over the exterior strip width and on the cantilevered portion of the sidewalk, shall be applied to the exterior strip. See 17.2.7 for the distribution of pedestrian live loads.

18.4.4 Minimum Slab Thickness Criteria

Check adequacy of chosen slab thickness by looking at live load deflection and dead load deflection (camber) criteria, using Service I Limit State.

18.4.4.1 Live Load Deflection Criteria

All concrete slab structures shall be designed to meet live load deflection limits **LRFD [2.5.2.6.2]**. Live load deflections for concrete slab structures are limited to L/1200, by the Bureau of Structures. The live load deflection, Δ_{LL+IM} , shall be calculated using factored loads described in 18.3.4.1 and 18.4.3.1 for Service I Limit State.

Place live loads in each design lane **LRFD [3.6.1.1.1]** and apply a multiple presence factor **LRFD [3.6.1.1.2]**. Use gross moment of inertia, I_g , based on entire slab width acting as a unit. Use modulus of elasticity E_c = 3800 ksi, see 18.2.2. The factored resistance, R_r , is described in 18.3.4.2.2.

Then check that, $\Delta_{LL+IM} \leq R_r$ is satisfied.

18.4.4.2 Dead Load Deflection (Camber) Criteria

All concrete slab structures shall be designed to meet dead load deflection (camber) limits **LRFD [5.6.3.5.2]**. Dead load deflections for concrete slab structures are computed using the gross moment of inertia, I_g . All dead loads are to be uniformly distributed across the width of the slab. These deflections are increased to provide for the time-dependent deformations of creep and shrinkage. Bureau of Structures currently calculates full camber as three times the dead load deflection. Most of the excess camber is dissipated during the first year of service, which is the time period that the majority of creep and shrinkage deflection occurs. Noticeable excess deflection or structure sag can normally be attributed to falsework settlement. Use modulus of elasticity $E_c = 3800$ ksi, see 18.2.2 . The dead load deflection, Δ_{DL} , shall be calculated using factored loads described in 18.3.4.1 and 18.4.2. The factored resistance, R_r , is described in 18.3.4.2.3.

WisDOT exception to AASHTO:

Calculating full camber as three times the dead load deflection, as stated in paragraph above, is an exception to **LRFD [5.6.3.5.2]**. This exception, used by the Bureau of Structures, is based on field observations using this method.

Then check that, $\Delta_{DL} \leq R_r$ is satisfied.

A "Camber Diagram" is shown in the plans on the "Superstructure" sheet. Provide camber values, as well as centerline and edge of slab elevations, at 0.1 points of all spans.

Simple-Span Concrete Slabs:

Maximum allowable camber for simple-span slabs is limited to 2 ½ inches. For simple-span slabs, Bureau of Structures practice indicates that using a minimum slab depth (ft) from the equation 1.1(S + 10) / 30, (where S is span length in feet), and meeting the live load deflection and dead load deflection (camber) limits stated in this section, provides an adequate slab section for most cases.

WisDOT exception to AASHTO:

The equation for calculating minimum slab depth for simple-spans, as stated in paragraph above, is an exception to **LRFD [Table 2.5.2.6.3-1]**. This exception, used by the Bureau of Structures, is based on past performance using this equation.

Continuous-Span Concrete Slabs:

Maximum allowable camber for continuous-span slabs is 1 ¾ inches.

18.4.5 Live Load Distribution

Live loads are distributed over an equivalent width, E, as calculated below. The equivalent distribution width applies for both live load moment and shear.

18.4.5.1 Interior Strip

Equivalent interior strip widths for slab bridges are covered in **LRFD [4.6.2.1.2, 4.6.2.3]**.

The live loads to be placed on these widths are axle loads (i.e., two lines of wheels) and the full lane load.

Where:

E = equivalent distribution width (ft)

Look at the distribution factor (for axle loads) calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

Compute the distribution factor associated with tributary portion of design lane load, to be applied to full lane load: **LRFD [3.6.1.2.4]**

$$DF = \frac{\left[\dfrac{(SWL)}{(10\text{ ft lane load width})}\right]}{(E)}$$

Where:

E = equivalent distribution width (ft)

SWL = Slab Width Loaded (with lane load) (ft) $\geq$ 0.

E – (distance from edge of slab to inside face of barrier) or

E – (distance from edge of slab to inside face of raised sidewalk)

Look at the distribution factor (for lane load) calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

18.4.6 Longitudinal Slab Reinforcement

The concrete cover on the top bars is 2 ½ inches, which includes a ½ inch wearing surface. The bottom bar cover is 1 ½ inches. Minimum clear spacing between adjacent longitudinal bars is 3 ½ inches. The maximum center to center spacing of adjacent bars shall not exceed 1.5 times the thickness of the slab or 18.0 inches **LRFD [5.10.3.2]**. When bundled bars are used, see **LRFD [5.10.3.1.5, 5.10.8.2.3, 5.10.8.4.2a]**.

18.4.6.1 Design for Strength

Strength Limit State considerations and assumptions are detailed in **LRFD [5.5.4, 5.6.2]**.

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, 5.6.1, 9.5.3]**

The area of longitudinal slab reinforcement, A_s , should be checked for fatigue stress range at locations where maximum stress range occurs along the structure, and for haunched slab structures, checked at the haunch/slab intercepts. The area should also be checked for fatigue stress range at bar reinforcement cutoff locations using Fatigue I Limit State. Check the reinforcement in an interior strip, where the largest number of fatigue cycles will occur.

Fatigue life of reinforcement is reduced by increasing the maximum stress level, bending of the bars and splicing of reinforcing bars by welding.

In regions where stress reversal takes place, continuous concrete slabs will be doubly reinforced. At these locations, the full stress range in the reinforcing bars from tension to compression is considered.

In regions of compressive stress due to unfactored permanent loads, fatigue shall be considered only if this compressive stress is less than 1.75 times the maximum tensile live load stress from the fatigue truck. The section properties for fatigue investigations shall be based on cracked sections where the sum of stresses, due to unfactored permanent loads, and 1.75 times the fatigue load is tensile and exceeds $0.095\ (f'_c)^{½}$.

The factored stress range, Q, shall be calculated using factored loads described in 18.3.5.1. The factored resistance, R_r , shall be calculated as in 18.3.5.2.1.

Then check that, Q (factored stress range) $\leq R_r$ is satisfied.

Reference is made to the design example in 18.5 of this chapter for computations relating to reinforcement remaining in tension throughout the fatigue cycle, or going through tensile and compressive stresses during the fatigue cycle.

18.4.6.3 Check for Crack Control

Service Limit State considerations and assumptions are detailed in **LRFD [5.5.2, 5.6.1, 5.6.7]**.

The area of longitudinal slab reinforcement, A_s , should be checked for crack control at locations where maximum tensile stress occurs along the structure, and for haunched slab structures, checked at the haunch/slab intercepts. The area should also be checked for crack control at bar reinforcement cutoff locations using Service I Limit State. Check the reinforcement in an interior and exterior strip (edge beam).

The use of high-strength steels and the acceptance of design methods where the reinforcement is stressed to higher proportions of the yield strength, makes control of flexural cracking by proper reinforcing details more significant than in the past. The width of flexural cracks is proportional to the level of steel tensile stress, thickness of concrete cover over the bars, and spacing of reinforcement. Improved crack control is obtained when the steel reinforcement is well distributed over the zone of maximum concrete tension.

Crack control criteria shall be applied when the tension in the cross-section exceeds 80% of the modulus of rupture, f_r , specified in **LRFD [5.4.2.6]**, for Service I Limit State. The spacing of reinforcement, s, in the layer closest to the tension face shall satisfy:

$s \leq (700\ \gamma_e / \beta_s f_{ss}) - 2\ (d_c)$ (in) Bar spacing, s, need not be less than 5 in. for control of flexural cracking **LRFD [5.6.7]**

in which:

$\beta_s = 1 + (d_c) / 0.7\ (h - d_c)$

Where:

γ_e = 1.00 for Class 1 exposure condition (bottom reinforcement)

γ_e = 0.75 for Class 2 exposure condition (top reinforcement)

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto, (in). For top reinforcement, d_c , should not include the ½" wearing surface

f_{ss} = tensile stress in steel reinforcement (ksi) $\leq 0.6 f_y$; use factored loads described in 18.3.4.1 at the Service I Limit State, to calculate (f_{ss})

h = overall depth of the section (in)

18.4.6.4 Minimum Reinforcement Check

The area of longitudinal slab reinforcement, A_s , should be checked for minimum reinforcement requirement at locations along the structure **LRFD [5.6.3.3]**.

The amount of tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r , or moment capacity, at least equal to the lesser of:

M_{cr} (or) $1.33\ M_u$

$M_{cr} = \gamma_3\ (\ \gamma_1\ f_r\ )\ S = 1.1\ f_r\ (I_g\ /\ c)$; $S = I_g\ /\ c$

Where:

f_r = $0.24\ \lambda\ (f'c)^{1/2}$ modulus of rupture (ksi) **LRFD [5.4.2.6]**

γ_1 = 1.6 flexural cracking variability factor

γ_3 = 0.67 ratio of minimum yield strength to ultimate tensile strength; for A615 Grade 60 reinforcement

I_g = gross moment of Inertia (in^4)

c = effective slab thickness/2 (in)

M_u = total factored moment, calculated using factored loads described in 18.3.3.1 for Strength I Limit State

λ = concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

Select lowest value of [M_{cr} (or) $1.33\ M_u$] = M_L

The factored resistance, M_r , or moment capacity, shall be calculated as in 18.3.3.2.1.

Then check that, $M_L \leq M_r$ is satisfied.

18.4.6.5 Bar Cutoffs

One-half of the bar steel reinforcement required for maximum moment can be cut off at a point, where the remaining one-half has the moment capacity, or factored resistance, M_r , equal to the total factored moment, M_u , at that point. This is called the theoretical cutoff point.

Select tentative cutoff point at theoretical cutoff point or at a distance equal to the development length from the point of maximum moment, whichever is greater. The reinforcement is extended beyond this tentative point for a distance equal to the effective depth of the slab, 15 bar diameters, or 1/20 of the clear span, whichever is greater. This cutoff point is acceptable, if it satisfies fatigue and crack control criteria. The continuing bars must be fully developed at this point **LRFD [5.10.8.1.2a]**.

18.4.6.5.1 Positive Moment Reinforcement

At least one-third of the maximum positive moment reinforcement in simple-spans and one-fourth of the maximum positive moment reinforcement in continuous-spans is extended along the same face of the slab beyond the centerline of the support **LRFD [5.10.8.1.2b]**.

18.4.6.5.2 Negative Moment Reinforcement

For negative moment reinforcement, the second tentative cutoff point is at the point of inflection. At least one-third of the maximum negative moment reinforcement must extend beyond this point for a distance equal to the effective depth of the slab, 12 bar diameters, or 1/16 of the clear span, whichever is greater **LRFD [5.10.8.1.2c]**.

18.4.7 Transverse Slab Reinforcement

18.4.7.1 Distribution Reinforcement

Distribution reinforcement is placed transversely in the bottom of the slab, to provide for lateral distribution of concentrated loads **LRFD [5.12.2.1]**. The criteria for main reinforcement parallel to traffic is applied. The amount of distribution reinforcement is to be determined as a percentage of the main reinforcing steel required for positive moment as given by the following formula:

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

Where:

L = span length (ft)

The above formula is conservative when applied to slab structures. This specification was primarily drafted for the relatively thin slabs on stringers.

18.4.7.2 Reinforcement in Slab over Piers

If the concrete superstructure rests on a pier cap (with columns) or directly on columns, design of transverse slab reinforcement over the pier is required. A portion of the slab over the pier is designed as a continuous transverse slab member (beam) along the centerline of the substructure. The depth of the assumed section is equal to the depth of the slab or haunch when the superstructure rests directly on columns. When the superstructure rests on a pier cap and the transverse slab member and pier cap act as a unit, the section depth will include the slab or haunch depth plus the cap depth. For a concrete slab, the width of the transverse slab member is equal to one-half the center to center spacing between columns (or 8 foot maximum) for the positive moment zone. The width equals the diameter of the column plus 6 inches for negative moment zone when no pier cap is present. The width equals the cap width for negative moment zone when a pier cap is present. Reference is made to the design example in 18.5 of this chapter for computations relating to transverse reinforcement in slab over the piers.

18.4.8 Shrinkage and Temperature Reinforcement

Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete exposed to daily temperature changes and in structural mass concrete.

The area, A_s , of reinforcement per foot for shrinkage and temperature effects, on each face and in each direction shall satisfy: **LRFD [5.10.6]**

$$A_s \geq 1.30\,(b)\,(h)\,/\,2\,(b+h)\,(f_y) \quad \text{and} \quad 0.11 \leq A_s \leq 0.60$$

Where:

A_s	=	area of reinforcement in each direction and on 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 (ksi) $\leq$ 75 ksi

Shrinkage and temperature reinforcement shall not be spaced farther apart than 3.0 times the component thickness or 18 inches. For components greater than 36 inches thick, the spacing shall not exceed 12 inches.

All longitudinal reinforcement and transverse reinforcement in the slab must exceed required A_s (on each face and in each direction), and not exceed maximum spacing.

18.4.9 Shear Check of Slab

Slab bridges designed for dead load and (HL-93) live load moments in conformance with **LRFD [4.6.2.3]** may be considered satisfactory in shear **LRFD [5.12.2.1]**.

18.4.10 Longitudinal Reinforcement Tension Check

The tensile capacity check of longitudinal reinforcement on the flexural tension side of a member is detailed in **LRFD [5.7.3.5]**.

The area of longitudinal reinforcement (in bottom of slab), A_s , should be checked for tensile capacity at the abutments, for dead load and (HL-93) live load on interior and exterior strips. The reinforcement at these locations shall have the capacity to resist the tension in the reinforcement produced by shear.

The factored shear, V_u , shall be calculated using factored loads described in 18.3.3.1 for Strength I Limit State. The factored tension force, T_{fact} , from shear, to be resisted is from **LRFD [Eq'n. 5.7.3.5-2]**, where $V_s = V_p = 0$, is:

$$T_{fact} = [\ V_u / \phi_v\ ] \cot \theta$$

Assume a diagonal crack would start at the inside edge of the bearing area. Assume the crack angle, θ, is 35 degrees. Calculate the distance from the bottom of slab to center of tensile reinforcement. Determine the distance D_{crack} from the end of the slab to the point at which the diagonal crack will intersect the bottom longitudinal reinforcement. Find the development length, ℓ_d , from Table 9.9-2, Chapter 9.

The nominal tensile resistance, T_{nom} , of the longitudinal bars at the crack location is:

$$T_{nom} = A_s f_y [\ D_{crack} - (\text{end cover})] / \ell_d \leq A_s f_y$$

Then check that, $T_{fact} \leq T_{nom}$ is satisfied.

If the values for T_{fact} and T_{nom} are close, the procedure for determining the crack angle, θ, as outlined in **LRFD [5.7.3.4.2]** should be used.

18.4.11 Uplift Check

Check for uplift at the abutments for (HL-93) live loads **LRFD [C3.4.1, 5.5.4.3]**. Compare the factored dead load reaction to the factored live load reaction. The reactions shall be calculated using factored loads described in 18.3.3.1 for Strength I Limit State. Place (HL-93) live loads in each design lane **LRFD [3.6.1.1.1]** and apply a multiple presence factor **LRFD [3.6.1.1.2]**.

18.4.12 Deflection Joints and Construction Joints

The designer should locate deflection joints in sidewalks and parapets on concrete slab structures according to the Standard *Vertical Face Parapet 'A'* in Chapter 30.

Refer to Standards *Continuous Haunched Slab* and *Continuous Flat Slab* in Chapter 18, for recommended construction joint guidelines.

18.4.13 Reinforcement Tables

Table 18.4-3 applies to: Rectangular Sections with Tension Reinforcement only

- Reinforcement Yield Strength (f_y) = 60,000 psi
- Concrete Compressive Strength (f'_c) = 4,000 psi

R_u	ρ	R_u	ρ	R_u	ρ	R_u	ρ	R_u	ρ
117.9	0.0020	335.6	0.0059	537.1	0.0098	722.6	0.0137	892.0	0.0176
123.7	0.0021	340.9	0.0060	542.1	0.0099	727.2	0.0138	896.1	0.0177
129.4	0.0022	346.3	0.0061	547.1	0.0100	731.7	0.0139	900.2	0.0178
135.2	0.0023	351.6	0.0062	552.0	0.0101	736.2	0.0140	904.4	0.0179
141.0	0.0024	357.0	0.0063	556.9	0.0102	740.7	0.0141	908.5	0.0180
146.7	0.0025	362.3	0.0064	561.8	0.0103	745.2	0.0142	912.5	0.0181
152.4	0.0026	367.6	0.0065	566.7	0.0104	749.7	0.0143	916.6	0.0182
158.1	0.0027	372.9	0.0066	571.6	0.0105	754.2	0.0144	920.7	0.0183
163.8	0.0028	378.2	0.0067	576.5	0.0106	758.7	0.0145	924.8	0.0184
169.5	0.0029	383.5	0.0068	581.4	0.0107	763.1	0.0146	928.8	0.0185
175.2	0.0030	388.8	0.0069	586.2	0.0108	767.6	0.0147	932.8	0.0186
180.9	0.0031	394.1	0.0070	591.1	0.0109	772.0	0.0148	936.9	0.0187
186.6	0.0032	399.3	0.0071	595.9	0.0110	776.5	0.0149	940.9	0.0188
192.2	0.0033	404.6	0.0072	600.8	0.0111	780.9	0.0150	944.9	0.0189
197.9	0.0034	409.8	0.0073	605.6	0.0112	785.3	0.0151	948.9	0.0190
203.5	0.0035	415.0	0.0074	610.4	0.0113	789.7	0.0152	952.9	0.0191
209.1	0.0036	420.2	0.0075	615.2	0.0114	794.1	0.0153	956.8	0.0192
214.8	0.0037	425.4	0.0076	620.0	0.0115	798.4	0.0154	960.8	0.0193
220.4	0.0038	430.6	0.0077	624.8	0.0116	802.8	0.0155	964.7	0.0194
225.9	0.0039	435.8	0.0078	629.5	0.0117	807.2	0.0156	968.7	0.0195
231.5	0.0040	441.0	0.0079	634.3	0.0118	811.5	0.0157	972.6	0.0196
237.1	0.0041	446.1	0.0080	639.0	0.0119	815.8	0.0158	976.5	0.0197
242.7	0.0042	451.3	0.0081	643.8	0.0120	820.1	0.0159	980.4	0.0198
248.2	0.0043	456.4	0.0082	648.5	0.0121	824.5	0.0160	984.3	0.0199
253.7	0.0044	461.5	0.0083	653.2	0.0122	828.8	0.0161	988.2	0.0200
259.3	0.0045	466.6	0.0084	657.9	0.0123	833.1	0.0162	992.1	0.0201
264.8	0.0046	471.7	0.0085	662.6	0.0124	837.3	0.0163	996.0	0.0202
270.3	0.0047	476.8	0.0086	667.3	0.0125	841.6	0.0164	999.8	0.0203
275.8	0.0048	481.9	0.0087	671.9	0.0126	845.9	0.0165	1003.7	0.0204
281.3	0.0049	487.0	0.0088	676.6	0.0127	850.1	0.0166	1007.5	0.0205
286.8	0.0050	492.1	0.0089	681.3	0.0128	854.3	0.0167	1011.3	0.0206
292.2	0.0051	497.1	0.0090	685.9	0.0129	858.6	0.0168	1015.1	0.0207
297.7	0.0052	502.2	0.0091	690.5	0.0130	862.8	0.0169	1018.9	0.0208
303.1	0.0053	507.2	0.0092	695.1	0.0131	867.0	0.0170	1022.7	0.0209
308.6	0.0054	512.2	0.0093	699.7	0.0132	871.2	0.0171	1026.5	0.0210
314.0	0.0055	517.2	0.0094	704.3	0.0133	875.4	0.0172	1030.3	0.0211
319.4	0.0056	522.2	0.0095	708.9	0.0134	879.5	0.0173	1034.0	0.0212
324.8	0.0057	527.2	0.0096	713.5	0.0135	883.7	0.0174	1037.8	0.0213
330.2	0.0058	532.2	0.0097	718.1	0.0136	887.9	0.0175	----	----

Table 18.4-3
R_u (psi) vs. ρ

R_u = coefficient of resistance (psi) = $M_u / \phi\, b\, d_s^2$ ρ = reinforcement ratio = $A_s / b\, d_s$

Table 18.4-4 can be used to select bar size and bar spacing to provide an adequate area of reinforcement to meet design requirements.

Bar Size Number	Nominal Dia. Inches	4 1/2"	5"	5 1/2"	6"	6 1/2"	7"	7 1/2"	8"	8 1/2"	9"	10"	12"
4	0.500	0.52	0.47	0.43	0.39	0.36	0.34	0.31	0.29	0.28	0.26	0.24	0.20
5	0.625	0.82	0.74	0.67	0.61	0.57	0.53	0.49	0.46	0.43	0.41	0.37	0.31
6	0.750	1.18	1.06	0.96	0.88	0.82	0.76	0.71	0.66	0.62	0.59	0.53	0.44
7	0.875	1.60	1.44	1.31	1.20	1.11	1.03	0.96	0.90	0.85	0.80	0.72	0.60
8	1.000	2.09	1.88	1.71	1.57	1.45	1.35	1.26	1.18	1.11	1.05	0.94	0.79
9	1.128	--	2.40	2.18	2.00	1.85	1.71	1.60	1.50	1.41	1.33	1.20	1.00
10	1.270	--	3.04	2.76	2.53	2.34	2.17	2.02	1.90	1.79	1.69	1.52	1.27
11	1.410	--	3.75	3.41	3.12	2.88	2.68	2.50	2.34	2.21	2.08	1.87	1.56

Table 18.4-4
Area of Bar Reinf. (in^2 / ft) vs. Spacing of Bars (in)

18.5 Design Example

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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 Eighth Edition - 2017***)

E18-1.1 Structure Preliminary Data

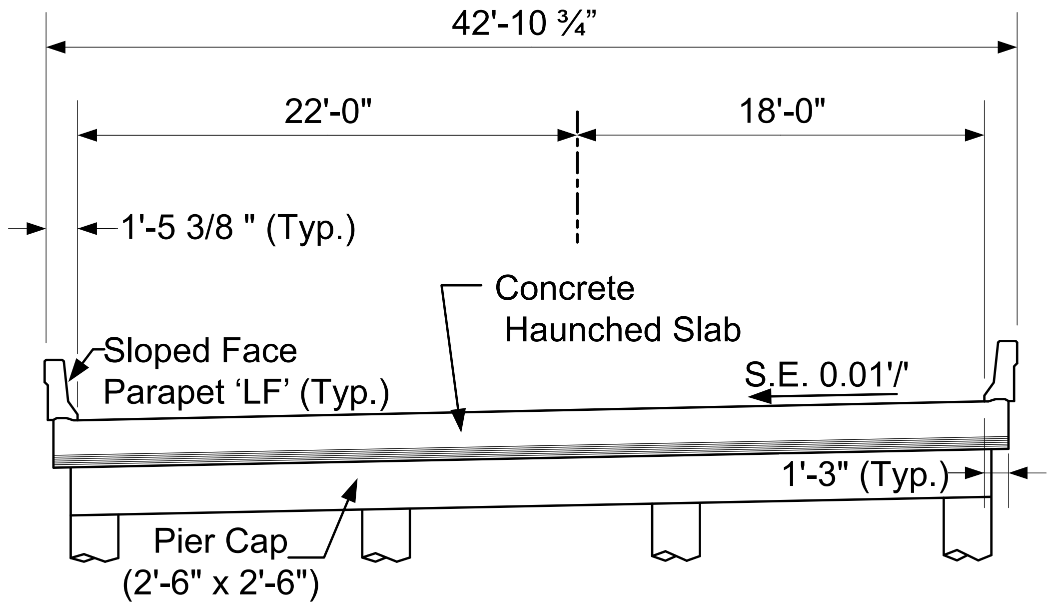


Figure E18.1

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.

		Strength I	Service I	Fatigue I
Load Factor (DC)	γ_{DC}	LRFD Table 3.4.1-2 0.90 (min.) 1.25 (max.)	LRFD Table 3.4.1-1 1.00	---
Load Factor (DW)	γ_{DW}	LRFD Table 3.4.1-2 0.65 (min.) 1.50 (max.)	LRFD Table 3.4.1-1 1.00	---
Load Factor (LL+IM)	γ_{LL+IM}	LRFD Table 3.4.1-1 1.75	LRFD Table 3.4.1-1 1.00	LRFD Table 3.4.1-1 1.75
Resistance Factor	ϕ	LRFD 5.5.4.2 0.90 flexure[1] 0.90 shear	LRFD 1.3.2.1 1.00	LRFD C1.3.2.1 1.00

Table E18.1
Load and Resistance Factors

[1] All reinforced concrete sections in this example were found to be tension-controlled sections as defined in **LRFD [5.6.2.1]**; therefore ϕ_f = 0.90

E18-1.3 Trial Slab Depth and Dead Loads (DC, DW)

Refer to Table 18.2-1 in 18.2.3 for an interior span length, L_2 , of 51 feet. The trial slab depth, d_{slab} (not including the 1/2 inch wearing surface), is estimated at:

$d_{slab} := 17$ in

The haunch depth, D_{haunch} , is approximately equal to d_{slab} divided by 0.6:

$$D_{haunch} := \frac{d_{slab}}{0.6} \rightarrow \frac{17}{0.6}$$

$D_{haunch} := \text{round}(D_{haunch})$ $D_{haunch} = 28$ in

D_{haunch} does not include the 1/2 inch wearing surface.

The length of the haunch, L_{haunch} , measured from the C/L of pier to the end of haunch, is approximately (0.15 to 0.18)*L_2. (L_2 equals interior span length = 51 feet)

$L_{hauchMin} := 0.15 \cdot L_2$ $L_{haunchMin} = 7.65$ ft

$L_{haunchMax} := 0.18 \cdot L_2$ $L_{haunchMax} = 9.18$ ft

Select the value for L_{haunch} to the nearest foot : $L_{haunch} = 8$ ft

The slab dead load, DC_{slab} , and the section properties of the slab, do not include the 1/2 inch wearing surface.

The dead load for the 17 inch slab depth, for a one foot design width, is calculated as follows:

$$DC_{17slab} := \frac{d_{slab}}{12} \cdot 1.0 \cdot w_c \rightarrow \frac{17}{12} \cdot 1.0 \cdot 150 \qquad \boxed{DC_{17slab} = 213} \text{ plf}$$

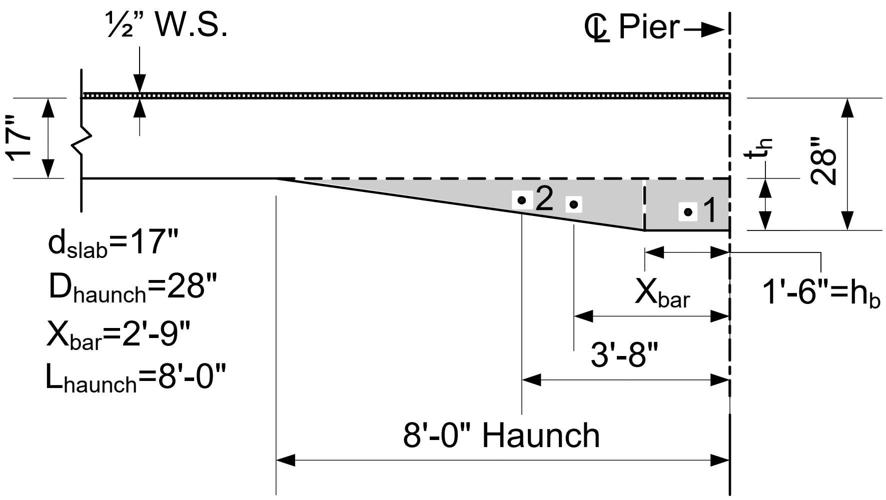


Figure E18.2

Haunched Section at Pier

For hand computations, determine the partial haunch dead load in the shaded area in Figure E18.2. Determine the center of gravity, X_{bar}, for this area and distribute its weight uniformly over twice this distance. Haunch dead load is often computed by computer programs.

The partial haunch thickness, t_h, equals:

$$t_h := D_{haunch} - d_{slab} \qquad \boxed{t_h = 11} \text{ in}$$

For a 2.5 ft. wide pier cap, the bottom width of the haunch is: $h_b := \frac{2.5}{2} + 0.25$ $\boxed{h_b = 1.5}$ ft

The area of sections (1 & 2) in Figure E18.2 and the location of their center of gravity is:

$$A_1 := h_b \cdot \frac{t_h}{12} \qquad A_2 := \frac{(L_{haunch} - h_b) \cdot \frac{t_h}{12}}{2} \qquad \boxed{A_1 = 1.38} \text{ ft}^2 \quad \boxed{A_2 = 2.98} \text{ ft}^2$$

$$X_{bar1} := \frac{h_b}{2} \qquad X_{bar2} := \frac{L_{haunch} - h_b}{3} + h_b \qquad \boxed{X_{bar1} = 0.75} \text{ ft} \quad \boxed{X_{bar2} = 3.67} \text{ ft}$$

The location of the center of gravity, X_{bar}, of the shaded area in Figure E18.2 is:

$$X_{bar} := \frac{A_1 \cdot X_{bar1} + A_2 \cdot X_{bar2}}{A_1 + A_2}$$ $\boxed{X_{bar} = 2.75}$ ft

The haunch dead load is uniformly distributed over a distance of $2 \cdot X_{bar} = 5.5$ feet.
For a one foot design width, its value is calculated as follows:

$$DC_{haunch} := \frac{A_1 + A_2}{2 \cdot X_{bar}} \cdot 1.0 \cdot w_c$$ $\boxed{DC_{haunch} = 119}$ plf

The dead load of the slab, DC_{slab} , is the total dead load from DC_{17slab} and DC_{haunch}.

The parapet dead load is uniformly distributed over the full width of the slab when designing for an interior strip of slab. The parapet dead load on a one foot design width, for an interior strip, is calculated as follows:

$$DC_{para} := \frac{2 \cdot w_{LF}}{slab_{width}} \rightarrow \frac{2 \cdot 387}{42.5}$$ $\boxed{DC_{para} = 18}$ plf

The parapet dead load is uniformly distributed over the exterior strip width of the slab when designing for an exterior strip (edge beam).

The 1/2 inch wearing surface dead load and a possible future wearing surface (FWS) dead load must also be included in the design of the slab. Therefore for a one foot design width:

$DC_{1/2"WS} = (0.5/12)(1.0)(w_c)$ $\boxed{DC_{1/2"WS} = 6}$ plf

$DW_{FWS} = (20)(1.0)$ $\boxed{DW_{FWS} = 20}$ plf

E18-1.4 Vehicular Live Load (LL) and Dynamic Load Allowance (IM)

The live load combinations used for design are:

LL#1:	Design Tandem (+ IM) + Design Lane Load	**LRFD [3.6.1.3.1]**
LL#2:	Design Truck (+ IM) + Design Lane Load	**LRFD [3.6.1.3.1]**
LL#3:	90% [Double Design Trucks (+ IM) + Design Lane Load]	**LRFD [3.6.1.3.1]**
LL#4:	Fatigue Truck (+ IM)	**LRFD [3.6.1.4.1]**
LL#5:	Design Truck (+ IM)	**LRFD [3.6.1.3.2]**
LL#6:	25% [Design Truck (+ IM)] + Design Lane Load	**LRFD [3.6.1.3.2]**

Table E18.2
Live Load Combinations

The live load combinations and dynamic load allowance, IM, **LRFD [3.6.2]** are applied to the Limit States as shown in Table E18.3.

Where (IM) is required, multiply the loads by (1 + IM/100) to include the dynamic effects of the load. (IM) is not applied to the Design Lane Load.

The live load force effect, Q_i , shall be taken as the largest from the live loads shown in Table E18.3 for that Limit State.

Strength I Limit State:	LL#1 , LL#2 , LL#3 [1]	IM = 33%
Service I Limit State: (for crack control criteria)	LL#1 , LL#2 , LL#3 [1]	IM = 33%
Service I Limit State: (for LL deflection criteria)	LL#5 , LL#6	IM = 33%
Fatigue I Limit State:	LL#4 (single Fatigue Truck)	IM = 15%

Table E18.3
Live Loads for Limit States

[1] (LL#3) is used to calculate negative live load moments between points of contraflexure and also reactions at interior supports. The points of contraflexure are located by placing a uniform load across the entire structure. For these moments and reactions, the results calculated from (LL#3) are compared with (LL#1) and (LL#2) results, and the critical value is selected.

E18-1.5 Minimum Slab Thickness Criteria

Check adequacy of chosen slab thickness by looking at live load deflection and dead load deflection (camber) criteria, using Service I Limit State.

E18-1.5.1 Live Load Deflection Criteria

All concrete slab structures shall be designed to meet live load deflection limits **LRFD [2.5.2.6.2]**, using Service I Limit State.

Looking at E18-1.2: $\eta_i := 1.0$ and from Table E18.1: $\gamma_{LLser1} := 1.0$ $\phi_{ser1} := 1.0$

$Q_i = \Delta_{LLser1}$ = largest live load deflection caused by live loads (LL#5 or LL#6)

See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM)

$$Q = \eta_i \cdot \gamma_{LLser1} \cdot \Delta_{LLser1} = (1.0) \cdot (1.0) \cdot \Delta_{LLser1}$$

Use (3) design lanes **LRFD [3.6.1.1.1]**, multiple presence of live load factor (m=0.85) **LRFD [3.6.1.1.2]** and gross moment of Inertia, I_g , based on the entire slab width acting as a unit, to calculate live load deflection. Use modulus of elasticity, E_c = 3800 ksi.

$R_n = \frac{L}{1200}$ = max. live load def'l. (L = span length)

$R_r = \phi_{ser1} \cdot R_n = 1.00 \cdot \frac{L}{1200}$

Therefore: $\Delta_{LLser1} \le \frac{L}{1200}$ (Limit States Equation)

The largest live load deflection is caused by live load (LL#5)

Span 1: $\Delta_{LLser1} = 0.29$ in $< \frac{L_1}{1200} = 0.38$ in O.K.

Span 2: $\Delta_{LLser1} = 0.47$ in $< \frac{L_2}{1200} = 0.51$ in O.K.

E18-1.5.2 Dead Load Deflection (Camber) Criteria

All concrete slab structures shall be designed to meet dead load deflection (camber) limits **LRFD [5.6.3.5.2]**, using Service I Limit State. Dead load deflections are computed using the gross moment of inertia, I_g. All dead loads are to be uniformly distributed across the slab width.

Looking at E18-1.2: $\eta_i := 1.0$

and from Table E18.1: $\gamma_{DCser1} := 1.0$ $\gamma_{DWser1} := 1.0$ $\phi_{ser1} := 1.0$

$Q_i = \Delta_{DL}$ = dead load deflection due to applied loads (DC, DW) as stated in E18-1.2.

$Q = \eta_i \cdot \gamma \cdot (\Delta_{DL}) = (1.0) \cdot (1.0) \cdot (\Delta_{DL})$

The Bureau of Structures currently calculates full camber as three times the dead load deflection. The maximum allowable camber for continuous spans is 1 3/4 inches (See 18.4.4.2). Therefore, the allowable dead load deflection is 1/3 of the maximum allowable camber. Use modulus of elasticity, E_c = 3800 ksi.

R_n = (max. allowable camber)/3 = 1 3/4 inches /3 = 0.583 inches

$R_r = \phi_{ser1} \cdot R_n = 1.00 \cdot (0.583) = 0.583$ in

Therefore: $\Delta_{DL} \le 0.583$ in (Limit States Equation)

Δ_{DL} (at 0.4 pt Span 1) = 0.17 in < 0.583 in O.K.

Δ_{DL} (at C/L of Span 2) = 0.27 in < 0.583 in O.K.

E18-1.6 Live Load Distribution (Interior Strip)

Live loads are distributed over an equivalent width, E, as calculated below. Equivalent strip widths for slab bridges are covered in **LRFD [4.6.2.1.2, 4.6.2.3].** The live loads to be placed on these widths are axle loads (i.e., two lines of wheels) and the full lane load. The equivalent distribution width applies for both live load moment and shear.

Single - Lane Loading: $E = 10.0 + 5.0\cdot(L_1\cdot W_1)^{0.5}$

Multi - Lane Loading: $E = 84.0 + 1.44\cdot(L_1\cdot W_1)^{0.5} \le 12.0\cdot\frac{W}{N_L}$

Where:

E = equivalent distribution width (in)

L_1 = modified span length taken equal to the lesser of the actual span or 60.0 ft (ft)

W_1 = modified edge to edge width of bridge taken to be equal to the lesser of the actual width or 60.0 ft for multi-lane loading, or 30.0 ft for single-lane loading (ft)

W = physical edge to edge width of bridge (ft)

N_L = number of design lanes as specified in **LRFD [3.6.1.1.1]**

For single-lane loading:

(Span 1, 3) $E := 10.0 + 5.0\cdot(38\cdot 30)^{0.5}$ $\boxed{E = 178}$ in

(Span 2) $E := 10.0 + 5.0\cdot(51\cdot 30)^{0.5}$ $\boxed{E = 205}$ in

For multi-lane loading:

$$12.0\cdot\frac{W}{N_L} = 12.0\cdot\frac{42.5}{3} = 170 \text{ in}$$

(Span 1, 3) $E := 84.0 + 1.44\cdot(38\cdot 42.5)^{0.5}$ $\boxed{E = 141}$ in < 170 in O.K.

(Span 2) $E := 84.0 + 1.44\cdot(51\cdot 42.5)^{0.5}$ $\boxed{E = 151}$ in < 170 in O.K.

E18-1.6.1 Strength and Service Limit State

Use the smaller equivalent widths, which are from multi-lane loading, when (HL-93) live load is to be distributed, for Strength I Limit State and Service I Limit State.

The distribution factor (DF) is computed for a design slab width equal to one foot.

$$DF = \frac{1}{E} \quad \text{(where E is in feet)}$$

The multiple presence factor (m) has been included in the equations for distribution width (E) and therefore aren't used to adjust the distribution factor (DF) **LRFD [3.6.1.1.2]**.

For spans 1 & 3: (E = 141" = 11.75')

$$DF := \frac{1}{11.75} \qquad \boxed{DF = 0.0851} \ \frac{\text{lanes}}{\text{ft}-\text{slab}}$$

For span 2: (E =151" = 12.583')

$$DF := \frac{1}{12.583} \qquad \boxed{DF = 0.0795} \ \frac{\text{lanes}}{\text{ft}-\text{slab}}$$

Look at the distribution factor calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

Therefore, <u>use DF = 0.0851 lanes/ft-slab</u> for all spans.

E18-1.6.2 Fatigue Limit State

Use equivalent widths from single-lane loading to check fatigue stress range criteria. For the Fatigue Limit State only one design truck (Fatigue Truck) is present **LRFD [3.6.1.4]**. Calculate the distribution factor (DF) and divide it by (1.20) to remove the effects of the multiple presence factor (m), which are present in the equation for equivalent width (E) **LRFD [3.6.1.1.2]**.

The distribution factor (DF) is computed for a <u>design slab width</u> equal to <u>one foot</u>.

$$DF = \frac{1}{E \cdot (1.20)} \qquad \text{(where E is in ft)}$$

For spans 1 & 3: (E = 178" = 14.833')

$$DF := \frac{1}{(14.833) \cdot (1.20)} \qquad \boxed{DF = 0.0562} \ \frac{\text{lanes}}{\text{ft}-\text{slab}}$$

For span 2: (E =205" = 17.083')

$$DF := \frac{1}{(17.083) \cdot (1.20)} \qquad \boxed{DF = 0.0488} \ \frac{\text{lanes}}{\text{ft}-\text{slab}}$$

Look at the distribution factor calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

Therefore, <u>use DF = 0.0562 lanes/ft.-slab</u> for all spans.

Table E18.4 Unfactored Moments (kip - ft) (on a one foot design width) **Interior Strip**

Point	M_{DC}[1]	M_{DW}[2]	DF=0.0851 (IM not used) +Design Lane	DF=0.0851 (IM not used) -Design Lane	DF=0.0851 (incl. IM=33%) +Design Tandem	DF=0.0851 (incl. IM=33%) -Design Tandem
0.1	9.6	0.8	3.2	-1.0	17.2	-3.2
0.2	15.9	1.3	5.5	-1.9	29.0	-6.4
0.3	18.7	1.6	7.1	-2.9	35.5	-9.6
0.4	18.1	1.5	7.9	-3.8	37.5	-12.8
0.5	14.1	1.2	7.9	-4.8	36.2	-16.0
0.6	6.6	0.6	7.2	-5.7	31.9	-19.2
0.7	-4.2	-0.4	5.6	-6.6	24.7	-22.3
0.789	-17.1	-1.5	3.7	-7.6	16.8	-25.1
0.8	-18.5	-1.6	3.5	-7.8	15.8	-25.5
0.9	-36.5	-3.1	2.4	-10.8	8.4	-28.7
1.0	-59.2	-4.9	2.2	-15.5	9.2	-31.9
1.1	-29.8	-2.5	1.9	-8.8	7.6	-21.8
1.157	-16.9	-1.4	2.3	-6.2	13.8	-19.8
1.2	-8.1	-0.7	2.9	-4.9	18.9	-18.4
1.3	7.2	0.6	5.4	-3.8	28.9	-14.9
1.4	16.4	1.4	7.5	-3.8	35.4	-11.4
1.5	19.6	1.6	8.2	-3.8	37.4	-8.0

Point	DF=0.0851 (incl. IM=33%) +Design Truck	DF=0.0851 (incl. IM=33%) -Design Truck	DF=0.0851 [3] (IM not used) (90%) of -Design Lane	DF=0.0851 [3] (incl. IM=33%) (90%) of -Double Design Trucks	DF=0.0562 (incl. IM=15%) +Fatigue Truck	DF=0.0562 (incl. IM=15%) -Fatigue Truck
0.1	18.1	-3.9	---	---	7.7	-1.4
0.2	29.3	-7.7	---	---	12.9	-2.8
0.3	34.4	-11.6	---	---	15.8	-4.2
0.4	35.4	-15.4	---	---	16.7	-5.5
0.5	33.9	-19.3	---	---	16.0	-6.9
0.6	30.7	-23.1	---	---	14.3	-8.4
0.7	23.3	-27.0	-6.0	-24.3	11.3	-9.8
0.789	14.0	-30.5	-6.9	-27.4	7.8	-11.0
0.8	13.0	-30.9	-7.0	-27.8	7.5	-11.2
0.9	9.0	-34.7	-9.7	-31.4	3.9	-16.0
1.0	10.1	-39.9	-13.9	-35.0	3.9	-23.0
1.1	8.0	-23.8	-8.0	-22.6	4.6	-13.6
1.157	12.1	-21.7	-5.6	-20.2	6.9	-9.0
1.2	15.3	-20.1	-4.4	-18.5	8.7	-7.7
1.3	27.7	-16.4	---	---	13.1	-6.3
1.4	35.4	-12.5	---	---	15.9	-4.8
1.5	37.2	-8.8	---	---	16.7	-3.4

Superscripts for Table E18.4 are defined on the following page.

In Table E18.4:

1 M_{DC} is moment due to slab dead load (DC_{slab}), parapet dead load (DC_{para}) after its weight is distributed across width of slab, and 1/2 inch wearing surface ($DC_{1/2"WS}$).

2 M_{DW} is moment due to future wearing surface (DW_{FWS}).

3 The points of contraflexure are located at the (0.66 pt.) of span 1 and the (0.25 pt.) of span 2, when a uniform load is placed across the entire structure. Negative moments in these columns are shown between the points of contraflexture per **LRFD [3.6.1.3.1]**.

E18-1.7 Longitudinal Slab Reinforcement (Interior Strip)

Select longitudinal reinforcement for an Interior Strip.

The concrete cover on the top bars is 2 1/2 inches, which includes a 1/2 inch wearing surface. The bottom bar cover is 1 1/2 inches. (See 18.4.6)

E18-1.7.1 Positive Moment Reinforcement for Span 1

Examine the 0.4 point of span 1

E18-1.7.1.1 Design for Strength

Design reinforcement using Strength I Limit State and considerations and assumptions detailed in **LRFD [5.5.4, 5.6.2]**

Looking at E18-1.2: $\eta_i := 1.0$

and from Table E18.1: $\gamma_{DCmax} := 1.25$ $\gamma_{DWmax} := 1.50$ $\gamma_{LLstr1} := 1.75$ $\phi_f := 0.9$

$Q_i = M_{DC}, M_{DW}, M_{LL+IM}$ **LRFD [3.6.1.2, 3.6.1.3.3]**; moments due to applied loads as stated in E18-1.2

$$Q = M_u = \eta_i\,[\gamma_{DCmax}(M_{DC}) + \gamma_{DWmax}(M_{DW}) + \gamma_{LLstr1}(M_{LL+IM})]$$

$$= 1.0\,[1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM})]$$

$$R_n = M_n = A_s \cdot f_s \cdot \left(d_s - \frac{a}{2}\right) \qquad \text{(See 18.3.3.2.1)}$$

$$M_r = \phi_f \cdot M_n = 0.90 \cdot A_s \cdot f_s \cdot \left(d_s - \frac{a}{2}\right)$$

Therefore : $M_u \le M_r$ (Limit States Equation)

$$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM}) \le 0.90\, A_s\, f_s\, (d_s - a/2)$$

The positive live load moment shall be the largest caused by live loads (LL#1 or LL#2). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM)

From Table E18.4, the largest live load moment is from (LL#1), therefore at (0.4 pt. - span 1):

M_{DC} = 18.1 kip-ft M_{DW} = 1.5 kip-ft M_{LL+IM} = 7.9 + 37.5 = 45.4 kip-ft

$M_u := 1.25\cdot(18.1) + 1.50\cdot(1.5) + 1.75\cdot(45.4)$ $\boxed{M_u = 104.3}$ kip-ft

$b := 12$ inches (for a one foot design width)

$d_s = d_{slab}$ - bott. bar clr. - 1/2 bott. bar dia.

$d_s := 17 - 1.5 - 0.6$ $\boxed{d_s = 14.9}$ in

Calculate R_u, coefficient of resistance:

$R_u = \dfrac{M_u}{\phi_f \cdot b \cdot d_s^2}$ $R_u := \dfrac{104.3\cdot(12)\cdot 1000}{0.9\cdot(12)\cdot 14.9^2}$ $\boxed{R_u = 522}$ psi

Solve for ρ, reinforcement ratio, using Table 18.4-3 (R_u vs ρ) in 18.4.13;

$\rho := 0.0095$

$A_s = \rho\cdot(b)\cdot d_s$ $A_s := 0.0095\cdot(12)14.9$ $\boxed{A_s = 1.7}$ $\dfrac{in^2}{ft}$

Try: #9 at 7" c-c spacing (A_s = 1.71 in^2/ft) from Table 18.4-4 in 18.4.13

Calculate the depth of the compressive stress block.

Assume $f_s = f_y$ (See 18.3.3.2.1) ; for f'_c = 4.0 ksi : $\alpha_1 := 0.85$ and $\beta_1 = 0.85$

$a = \dfrac{A_s \cdot f_y}{\alpha_1 \cdot f'_c \cdot b}$ $a := \dfrac{1.71\cdot(60)}{0.85\cdot(4.0)\cdot 12}$ $\boxed{a = 2.51}$ in

If $\dfrac{c}{d_s} \le 0.6$ for (f_y= 60 ksi) **LRFD [5.6.2.1]**, then reinforcement has yielded and the assumption is correct.

$\beta_1 := 0.85$ $c := \dfrac{a}{\beta_1}$ $\boxed{c = 2.96}$ in

$\dfrac{c}{d_s} = 0.2 < 0.6$ therefore, the reinforcement will yield.

$M_r = 0.90\cdot A_s \cdot f_y \cdot \left(d_s - \dfrac{a}{2}\right)$

$M_r := 0.9\cdot(1.71)\cdot 60.0\cdot\left(\dfrac{14.9 - \dfrac{2.51}{2}}{12}\right)$ $\boxed{M_r = 105}$ kip-ft

Therefore, $M_u = 104.3$ kip-ft < $M_r = 105$ kip-ft O.K.

E18-1.7.1.2 Check for Fatigue

Check reinforcement using Fatigue I Limit State and considerations and assumptions detailed in **LRFD [5.5.3, 5.6.1, 9.5.3]**.

Looking at E18-1.2: $\eta_i := 1.0$ and from Table E18.1: $\gamma_{LLfatigue} := 1.75$ $\phi_{fatigue} := 1.0$

When reinforcement remains in tension throughout the fatigue cycle,

$Q_i = \Delta f = f_{range}$ = stress range in bar reinforcement due to flexural moment range (M_{range}) caused by Fatigue Truck (LL#4). See Table E18.2 and E18.3 in E18-1.4 for description of live load and dynamic load allowance (IM)

$$Q = \eta_i \cdot \gamma_{LLfatigue} \cdot f_{range} = (1.0)\cdot(1.75)\cdot f_{range}$$

$$R_n = \left(\Delta F_{TH}\right) = 26 - 0.37 \cdot f_{min}$$ for $f_y = 60$ ksi (See 18.3.5.2.1)

$$R_r = \phi_{fatigue} \cdot R_n = 1.0 \cdot \left(26 - 0.37 \cdot f_{min}\right)$$

Therefore: $1.75 \cdot \left(f_{range}\right) \le 26 - 0.37 \cdot f_{min}$ (Limit States Equation)

From Table E18.4, the moments at (0.4 pt.) of span 1 are:

$M_{DC} = 18.1$ kip-ft $M_{DW} = 1.5$ kip-ft

+Fatigue Truck = 16.7 kip-ft -Fatigue Truck = -5.5 kip-ft

In regions of tensile stress due to permanent loads, fatigue criteria should be checked.

The section properties for fatigue shall be based on cracked sections where the sum of stresses, due to unfactored permanent loads, and ($\gamma_{LLfatigue} = 1.75$) times the fatigue load is tensile and exceeds $0.095\sqrt{f'_c}$ **LRFD [5.5.3.1]**

Allowable tensile stress for fatigue (cracking stress):

$$f_{tensile} = 0.095\sqrt{f'_c} = 0.095 \cdot \sqrt{4}$$ $\boxed{f_{tensile} = 0.19}$ ksi

Calculate fatigue moment and then select section properties:

$$M_{fatigue} = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.75(\text{Fatigue Truck})$$

$M_{fatigueMax} := 1.0 \cdot (18.1) + 1.0(1.5) + 1.75(16.7)$ $\boxed{M_{fatigueMax} = 48.83}$ kip-ft (tension)

$M_{fatigueMin} := 1.0 \cdot (18.1) + 1.0(1.5) + 1.75(-5.5)$ $\boxed{M_{fatigueMin} = 9.98}$ kip-ft (tension)

Calculate stress due to $M_{fatigue}$: $f_{fatigue} = \frac{M_{fatigue} \cdot (y)}{I_g}$

$y = \frac{d_{slab}}{2} = \frac{17}{2}$ $\boxed{y = 8.5}$ in

$I_g = \frac{1}{12} \cdot b \cdot d_{slab}^3 = \frac{1}{12} \cdot (12) \, 17^3$ $\boxed{I_g = 4913}$ in^4

$f_{fatigueMax} := \frac{M_{fatigueMax} \cdot (y) \cdot 12}{I_g}$ $\boxed{f_{fatigueMax} = 1.01}$ ksi (tension) > $f_{tensile}$ (0.190 ksi)

$f_{fatigueMin} := \frac{M_{fatigueMin} \cdot (y) \cdot 12}{I_g}$ $\boxed{f_{fatigueMin} = 0.21}$ ksi (tension) > $f_{tensile}$ (0.190 ksi)

Values of $f_{fatigue}$ exceed $f_{tensile}$ during the fatigue cycle, therefore analyze fatigue using cracked section properties.

Looking at values of $M_{fatigue}$, shows that the reinforcement remains in tension throughout the fatigue cycle. Therefore:

M_{range} = (+ Fatigue Truck) - (-Fatigue Truck)

$M_{range} := 16.7 - (-5.5)$ $\boxed{M_{range} = 22.2}$ kip-ft

The moment arm used in equations below is: (j) (d_s) Therefore, using:

$A_s = 1.7 \frac{in^2}{ft}$ (required for strength), $d_s = 14.9$ in , $n := 8$, and transformed section analysis, gives a value of $j := 0.893$

$f_{range} = \frac{M_{range}}{A_s \cdot (j) \cdot d_s} = \frac{22.2 \cdot 12}{1.7 \cdot (0.893) 14.9}$ $\boxed{f_{range} = 11.78}$ ksi

$f_{range1.75} := 1.75 \cdot f_{range}$ $\boxed{f_{range1.75} = 20.61}$ ksi

$f_{min} = \frac{M_{DC} + M_{DW} + 1.75(-FatigueTruck)}{A_s \cdot (j) \cdot d_s}$

$f_{min} := \frac{[18.1 + 1.5 + 1.75 \cdot (-5.5)] \cdot 12}{1.7 \cdot (0.893) 14.9}$ $\boxed{f_{min} = 5.29}$ ksi

$R_r := 26 - 0.37 \cdot f_{min}$ $\boxed{R_r = 24.04}$ ksi

Therefore, $1.75 \cdot (f_{range}) = 20.61$ ksi < $R_r = 24.04$ ksi O.K.

E18-1.7.1.3 Check Crack Control

Check reinforcement using Service I Limit State and considerations and assumptions detailed in **LRFD [5.5.2, 5.6.1, 5.6.7]**

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**; λ = 1.0 (normal wgt. conc.)**LRFD [5.4.2.8]**

$f_r = 0.24 \cdot \lambda \sqrt{f'_c}$ $\boxed{f_r = 0.48}$ ksi $f_{r80\%} := 0.8 \cdot f_r$ $\boxed{f_{r80\%} = 0.38}$ ksi

$$f_T = \frac{M_s \cdot (c)}{I_g}$$

$c := \dfrac{d_{slab}}{2}$ $\boxed{c = 8.5}$ in $I_g := \dfrac{1}{12} \cdot b \cdot d_{slab}{}^3$ $\boxed{I_g = 4913}$ in^4

Looking at E18-1.2: $\eta_i := 1.0$

and from Table E18.1: $\gamma_{DC.ser1} := 1.0$ $\gamma_{DW.ser1} := 1.0$ $\gamma_{LLser1} := 1.0$ $\phi_{ser1} := 1.0$

$Q_i = M_{DC}, M_{DW}, M_{LL+IM}$ **LRFD [3.6.1.2, 3.6.1.3.3]**; moments due to applied loads as stated in E18-1.2

$Q = M_S = \eta_i \,[\gamma_{DC.ser1}(M_{DC}) + \gamma_{DW.ser1}(M_{DW}) + \gamma_{LLser1}(M_{LL+IM})]$

$= 1.0\,[1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})]$

Therefore, M_s becomes:

$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$ (Factored Load Equation)

Using same moments selected from Table E18.4 for Strength Design in E18-1.7.1.1, at (0.4 pt.) of span 1, provides:

M_{DC} = 18.1 kip-ft M_{DW} = 1.5 kip-ft M_{LL+IM} = 7.9 + 37.5 = 45.4 kip-ft (LL#1)

$M_s := 1.0 \cdot (18.1) + 1.0 \cdot (1.5) + 1.0 \cdot (45.4)$ $\boxed{M_s = 65}$ kip-ft

$f_T = \dfrac{M_s \cdot (c)}{I_g}$ $f_T := \dfrac{65.0 \cdot (8.5) \cdot 12}{4913}$ $\boxed{f_T = 1.35}$ ksi

f_T = 1.35 ksi > 80% f_r = 0.38 ksi; therefore, check crack control criteria

Knowing $A_s = 1.7 \dfrac{in^2}{ft}$ (required for strength)

Try: #9 at 7" c-c spacing (A_s = 1.71 in^2/ft) from Table 18.4-4 in 18.4.13

The spacing (s) of reinforcement in the layer closest to the tension face shall satisfy:

$$s \le \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot (d_c) \qquad \text{in which:} \quad \beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)}$$

$\gamma_e := 1.00$ for Class 1 exposure condition (bottom reinforcement)

d_c = clr. cover + 1/2 bar dia.
= thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in). See Figure E18.3

$$d_c := 1.5 + \frac{1.128}{2} \qquad \boxed{d_c = 2.064} \text{ in}$$

h = overall depth of the section (in). See Figure E18.3

$$h := d_{slab} \qquad \boxed{h = 17} \text{ in}$$

$$\beta_s := 1 + \frac{d_c}{0.7 \cdot (h - d_c)} \qquad \boxed{\beta_s = 1.2}$$

f_{ss} = tensile stress in steel reinforcement at the Service I Limit State (ksi) $\le 0.6\ f_y$

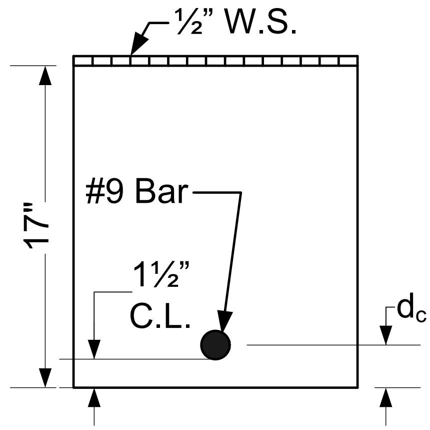


Figure E18.3

Cross Section - (0.4 pt.) Span 1

The moment arm used in the equation below to calculate f_{ss} is: (j) (h - d_c)
As shown in fatigue calculations in E18-1.7.1.2, j = 0.893

$$f_{ss} = \frac{M_s}{A_s \cdot (j) \cdot (h - d_c)} = \frac{65.0 \cdot (12)}{1.71 \cdot (0.893)(17 - 2.064)} \qquad \boxed{f_{ss} = 34.2} \text{ ksi} \quad \le 0.6\ f_y \text{ O.K.}$$

$$s \le \frac{700 \cdot (1.00)}{1.2 \cdot (34.2)} - 2 \cdot (2.064) = 17.0 - 4.1 = 12.9 \quad \text{in}$$

$s \leq 12.9$ in

Therefore, spacing prov'd. = 7 in < 12.9 in O.K.

Use: #9 at 7" c-c spacing in span 1 (Max. positive reinforcement).

E18-1.7.1.4 Minimum Reinforcement Check

The amount of tensile reinforcement shall be adequate to develop a factored flexural resistance (M_r), or moment capacity, at least equal to the lesser of: **LRFD[5.6.3.3]**

M_{cr} (or) $1.33 M_u$

$$M_{cr} = \gamma_3 (\gamma_1 \cdot f_r) S \qquad \text{where: } S = \frac{I_g}{c} \qquad \text{therefore, } M_{cr} = 1.1(f_r)\frac{I_g}{c}$$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

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

$f_r = 0.24\lambda\sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r = 0.24\sqrt{4}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.48}$ ksi

$I_g := \frac{1}{12} \cdot b \cdot d_{slab}^{\ 3}$ $\boxed{I_g = 4913}$ in^4 $c := \frac{d_{slab}}{2}$ $\boxed{c = 8.5}$ in

$$M_{cr} = \frac{1.1 f_r \cdot (I_g)}{c} = \frac{1.1 \cdot 0.48 \cdot (4913)}{8.5(12)}$$ $\boxed{M_{cr} = 25.43}$ kip-ft

$1.33 \cdot M_u = 138.75$ kip-ft , where M_u was calculated for Strength Design in E18-1.7.1.1 and (M_u = 104.3 kip-ft)

M_{cr} controls because it is less than 1.33 M_u

As shown in E18-1.7.1.1, the reinforcement yields, therefore:

$$M_r = 0.90 \cdot A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right)$$ $\boxed{M_r = 105}$ kip-ft

Therefore, M_{cr} = 25.43 kip-ft < M_r = 105 kip-ft O.K.

E18-1.7.2 Negative Moment Reinforcement at Piers

Examine at C/L of Pier

E18-1.7.2.1 Design for Strength

Following the procedure in E18-1.7.1.1, using Strength I Limit State:

$$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM}) \leq 0.90\, A_s\, f_s\, (d_s - a/2)$$

The negative live load moment shall be the largest caused by live loads (LL#1, LL#2 or LL#3). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM).

From Table E18.4, the largest live load moment is from (LL#2), therefore at (C/L of Pier):

M_{DC} = -59.2 kip-ft M_{DW} = -4.9 kip-ft M_{LL+IM} = -15.5 + (-39.9) = -55.4 kip-ft

$M_u := 1.25\cdot(-59.2) + 1.50\cdot(-4.9) + 1.75\cdot(-55.4)$ $\boxed{M_u = -178.3}$ kip-ft

$b := 12$ inches (for a one foot design width) and $\boxed{d_s = 25.4}$ in

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_s, are:

$\boxed{R_u = 307.1}$ psi $\boxed{\rho = 0.0054}$ $\boxed{A_s = 1.65}$ $\frac{in^2}{ft}$

Try: #8 at 5 1/2" c-c spacing (A_s = 1.71 in^2/ft) from Table 18.4-4 in 18.4.13

Assume $f_s = f_y$, then the depth of the compressive stress block is: $\boxed{a = 2.51}$ in

Then, $\boxed{c = 2.96}$ in and $\frac{c}{d_s} = 0.12 < 0.6$ therefore, the reinforcement will yield.

The factored resistance is: $\boxed{M_r = 186.6}$ kip-ft

Therefore, M_u = 178.3 kip-ft < M_r = 186.6 kip-ft O.K.

E18-1.7.2.2 Check for Fatigue

Following the procedure in E18-1.7.1.2, using Fatigue I Limit State:

$$1.75\cdot\left(f_{range}\right) \leq 26 - 0.37\cdot f_{min} \qquad \text{(for } f_y = 60 \text{ ksi)}$$

From Table E18.4, the moments at (C/L Pier) are:

M_{DC} = -59.2 kip-ft M_{DW} = -4.9 kip-ft

+Fatigue Truck = 3.9 kip-ft -Fatigue Truck = -23.0 kip-ft

In regions of tensile stress due to permanent loads, fatigue criteria should be checked.

Allowable tensile stress for fatigue (cracking stress): $\boxed{f_{tensile} = 0.19}$ ksi

Calculate fatigue moment and then select section properties:

$M_{fatigue}$ = 1.0(M_{DC}) + 1.0(M_{DW}) +1.75(Fatigue Truck)

$\boxed{M_{fatigueMax} = -104.35}$ kip-ft (tension) $\boxed{M_{fatigueMin} = -57.3}$ kip-ft (tension)

Calculate stress due to $M_{fatigue}$, where : $\boxed{y = 14}$ in $\boxed{I_g = 21952}$ in^4

$$f_{fatigue} = \frac{M_{fatigue} \cdot (y) \cdot 12}{I_g}$$

$\boxed{f_{fatigueMax} = 0.8}$ ksi (tension) > $f_{tensile}$ (0.190 ksi)

$\boxed{f_{fatigueMin} = 0.44}$ ksi (tension) > $f_{tensile}$ (0.190 ksi)

Values of $f_{fatigue}$ exceed $f_{tensile}$ during the fatigue cycle, therefore analyze fatigue using cracked section properties.

Looking at values of $M_{fatigue}$, shows that the reinforcement remains in tension throughout the fatigue cycle. Therefore:

M_{range} = (- Fatigue Truck) - (+Fatigue Truck) $\boxed{M_{range} = -26.9}$ kip-ft

The values for A_s, d_s, n and j (from transformed section) used to calculate f_{range} and f_{min} are:

$A_s = 1.65 \ \frac{in^2}{ft}$ (required for strength), $d_s = 25.4$ in , $n := 8$, $j := 0.915$

The values for f_{range}, $f_{range1.75}$, and f_{min} are:

$\boxed{f_{range} = 8.42}$ ksi $\boxed{f_{range1.75} = 14.73}$ ksi $\boxed{f_{min} = 17.92}$ ksi

The factored resistance is: $\boxed{R_r = 19.37}$ ksi

Therefore, $1.75 \cdot (f_{range})$ = 14.73 ksi < R_r = 19.37 ksi <u>O.K.</u>

E18-1.7.2.3 Check Crack Control

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**

Following the procedure in E18-1.7.1.3, using Service I Limit State:

$\boxed{f_r = 0.48}$ ksi $\boxed{f_{r80\%} = 0.38}$ ksi $\boxed{c = 14}$ in $\boxed{I_g = 21952}$ in^4

$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$

Using same moments selected from Table E18.4 for Strength Design in E18-1.7.2.1, at (C/L of Pier), provides:

M_{DC} = -59.2 kip-ft M_{DW} = -4.9 kip-ft M_{LL+IM} = -15.5 + (-39.9) = -55.4 kip-ft (LL#2)

$M_s := 1.0 \cdot (59.2) + 1.0 \cdot (4.9) + 1.0 \cdot (55.4)$ $\boxed{M_s = 119.5}$ kip-ft

$f_T = \dfrac{M_s \cdot c}{I_g}$ $f_T := \dfrac{119.5 \cdot (14) \cdot 12}{21952}$ $\boxed{f_T = 0.91}$ ksi

f_T =0.91 ksi > 80% f_r = 0.38 ksi; therefore, check crack control criteria

Knowing $A_s = 1.65 \dfrac{in^2}{ft}$ (required for strength)

Try: #8 at 5 1/2" c-c spacing (A_s = 1.71 in^2/ft) from Table 18.4-4 in 18.4.13

The values for γ_e , d_c , h , and β_s , used to calculate max. spacing (s) of reinforcement are :

$\gamma_e := 0.75$ for Class 2 exposure condition (top reinforcement)

$\boxed{d_c = 2.5}$ in (See Figure E18.4) $\boxed{h = 28}$ in (See Figure E18.4) $\boxed{\beta_s = 1.14}$

f_{ss} = tensile stress in steel reinforcement at the Service I Limit State (ksi) $\leq 0.6\ f_y$

The moment arm used to calculate f_{ss} is: (j) (h - d_c)
As shown in fatigue calculations in E18-1.7.2.2, j = 0.915

The value of f_{ss} and (s) are:

$\boxed{f_{ss} = 35.94}$ ksi $\leq 0.6\ f_y$ O.K. $s \leq \dfrac{700 \cdot (0.75)}{1.14 \cdot (35.94)} - 2 \cdot (2.50) = 12.8 - 5.0 = 7.8$ in

$s \leq 7.8$ in

Therefore, spacing prov'd. = 5 1/2 in < 7.8 in O.K.

To insure that the reinforcement has the moment capacity to handle the Wisconsin Standard Permit Vehicle (Wis-SPV), the spacing was reduced to 5 inches. (See E18-1.8)

Use: #8 at 5" c-c spacing at C/L Pier (Max. negative reinforcement), $A_s = 1.88 \dfrac{in^2}{ft}$

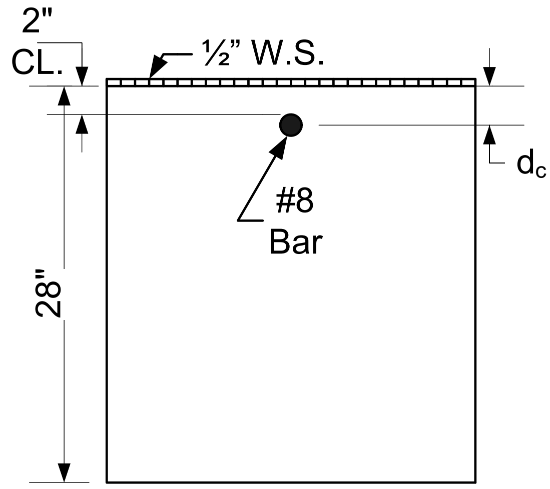


Figure E18.4

Cross Section - (at C/L of Pier)

E18-1.7.2.4 Minimum Reinforcement Check

The amount of tensile reinforcement shall be adequate to develop a factored flexural resistance (M_r), or moment capacity, at least equal to the lesser of: **LRFD [5.6.3.3]**

M_{cr} (or) $1.33M_u$

from E18-1.7.1.4, $M_{cr} = 1.1(f_r)\frac{I_g}{c}$

Where:

$f_r = 0.24\lambda\sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r = 0.24\sqrt{4}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.48}$ ksi

$I_g := \frac{1}{12} \cdot b \cdot D_{haunch}^{\ 3}$ $\boxed{I_g = 21952}$ in^4 $c := \frac{D_{haunch}}{2}$ $\boxed{c = 14}$ in

$M_{cr} = \frac{1.1 f_r \cdot (I_g)}{c} = \frac{1.1 \cdot 0.48 \cdot (21952)}{14(12)}$ $\boxed{M_{cr} = 68.99}$ kip-ft

$1.33 \cdot M_u = 237.1$ kip-ft , where M_u was calculated for Strength Design in E18-1.7.2.1 and (M_u = 178.3 kip-ft)

M_{cr} controls because it is less than 1.33 M_u

By examining E18-1.7.2.1, the reinforcement yields, therefore:

$$M_r = 0.90 \cdot A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right) \qquad \boxed{M_r = 204.1} \text{ kip-ft}$$

Therefore, $M_{cr} = 68.99$ kip-ft $<$ $M_r = 204.1$ kip-ft O.K.

E18-1.7.3 Positive Moment Reinforcement for Span 2

Examine the 0.5 point of span 2

E18-1.7.3.1 Design for Strength

Following the procedure in E18-1.7.1.1, using Strength I Limit State:

$$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM}) \leq 0.90\, A_s\, f_s\, (d_s - a/2)$$

The positive live load moment shall be the largest caused by live loads (LL#1 or LL#2). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM).

From Table E18.4, the largest live load moment is from (LL#1), therefore at (0.5 pt.) of span 2:

$M_{DC} = 19.6$ kip-ft $\qquad M_{DW} = 1.6$ kip-ft $\qquad M_{LL+IM} = 8.2 + 37.4 = 45.6$ kip-ft

$$M_u := 1.25 \cdot (19.6) + 1.50 \cdot (1.6) + 1.75 \cdot (45.6) \qquad \boxed{M_u = 106.7} \text{ kip-ft}$$

$b := 12$ inches (for a one foot design width) and $\boxed{d_s = 14.9}$ in

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_s, are:

$\boxed{R_u = 534}$ psi $\qquad \boxed{\rho = 0.0097} \qquad \boxed{A_s = 1.73}\ \frac{in^2}{ft}$

Try: #9 at 6" c-c spacing (A_s = 2.00 in^2/ft) from Table 18.4-4 in 18.4.13

Assume $f_s = f_y$, then the depth of the compressive stress block is: $\boxed{a = 2.94}$ in

Then, $\boxed{c = 3.46}$ in and $\dfrac{c}{d_s} = 0.23 < 0.6$ therefore, the reinforcement will yield.

The factored resistance is: $\boxed{M_r = 120.9}$ kip-ft

Therefore, $M_u = 106.7$ kip-ft $<$ $M_r = 120.9$ kip-ft O.K.

E18-1.7.3.2 Check for Fatigue

Following the procedure in E18-1.7.1.2, using Fatigue I Limit State:

$1.75 \cdot (f_{range}) \le 26 - 0.37 \cdot f_{min}$ (for f_y = 60 ksi)

From Table E18.4, the moments at (0.5 pt.) of span 2 are:

M_{DC} = 19.6 kip-ft M_{DW} = 1.6 kip-ft

+Fatigue Truck = 16.7 kip-ft -Fatigue Truck = -3.4 kip-ft

In regions of tensile stress due to permanent loads, fatigue criteria should be checked.

Allowable tensile stress for fatigue (cracking stress): $\boxed{f_{tensile} = 0.19}$ ksi

Calculate fatigue moment and then select section properties:

$M_{fatigue}$ = 1.0(M_{DC}) + 1.0(M_{DW}) +1.75(Fatigue Truck)

$\boxed{M_{fatigueMax} = 50.42}$ kip-ft (tension) $\boxed{M_{fatigueMin} = 15.3}$ kip-ft (tension)

Calculate stress due to $M_{fatigue}$, where $\boxed{y = 8.5}$ in $\boxed{I_g = 4913}$ in^4
:

$f_{fatigue} = \dfrac{M_{fatigue} \cdot (y) \cdot 12}{I_g}$

$\boxed{f_{fatigueMax} = 1.05}$ ksi (tension) > $f_{tensile}$ (0.190 ksi)

$\boxed{f_{fatigueMin} = 0.32}$ ksi (tension) > $f_{tensile}$ (0.190 ksi)

Values of $f_{fatigue}$ exceed $f_{tensile}$ during the fatigue cycle, therefore analyze fatigue using cracked section properties.

Looking at values of $M_{fatigue}$, shows that the reinforcement remains in tension throughout the fatigue cycle. Therefore:

M_{range} = (+ Fatigue Truck) - (-Fatigue Truck) $\boxed{M_{range} = 20.1}$ kip-ft

The values for A_s, d_s, n and j (from transformed section) used to calculate f_{range} and f_{min} are:

$A_s = 1.73 \ \dfrac{in^2}{ft}$ (required for strength), $d_s = 14.9$ in , $n := 8$, $j := 0.892$

The values for f_{range} , $f_{range1.75}$, and f_{min} are:

$\boxed{f_{range} = 10.43}$ ksi $\boxed{f_{range1.75} = 18.25}$ ksi $\boxed{f_{min} = 7.96}$ ksi

The factored resistance is: $\boxed{R_r = 23.06}$ ksi

Therefore, $1.75 \cdot (f_{range})$ = 18.25 ksi < R_r = 23.06 ksi <u>O.K.</u>

E18-1.7.3.3 Check Crack Control

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**

Following the procedure in E18-1.7.1.3, using Service I Limit State:

$\boxed{f_r = 0.48}$ ksi $\boxed{f_{r80\%} = 0.38}$ ksi $\boxed{c = 8.5}$ in $\boxed{I_g = 4913}$ in^4

$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$

Using same moments selected from Table E18.4 for Strength Design in E18-1.7.3.1, at (0.5 pt.) of span 2 provides:

M_{DC} = 19.6 kip-ft M_{DW} = 1.6 kip-ft M_{LL+IM} = 8.2 + 37.4 = 45.6 kip-ft (LL#1)

$M_s := 1.0\cdot(19.6) + 1.0\cdot(1.6) + 1.0\cdot(45.6)$ $\boxed{M_s = 66.8}$ kip-ft

$f_T = \dfrac{M_s \cdot c}{I_g}$ $f_T := \dfrac{66.8\cdot(8.5)\cdot 12}{4913}$ $\boxed{f_T = 1.39}$ ksi

f_T =1.39 ksi > 80% f_r = 0.38 ksi; therefore, check crack control criteria

Knowing $A_s = 1.73\ \dfrac{in^2}{ft}$ (required for strength)

Try: #9 at 6" c-c spacing (A_s = 2.00 in^2/ft) from Table 18.4-4 in 18.4.13

The values for γ_e , d_c , h , and β_s , used to calculate max. spacing (s) of reinforcement are :

$\gamma_e := 1.00$ for Class 1 exposure condition (bottom reinforcement)

$\boxed{d_c = 2.064}$ in (See Figure E18.5) $\boxed{h = 17}$ in (See Figure E18.5) $\boxed{\beta_s = 1.2}$

f_{ss} = tensile stress in steel reinforcement at the Service I Limit State (ksi) $\leq 0.6\ f_y$

The moment arm used to calculate f_{ss} is: (j) (h - d_c)
As shown in fatigue calculations in E18-1.7.3.2, j = 0.892

The value of f_{ss} and (s) are:

$\boxed{f_{ss} = 30.08}$ ksi $\leq 0.6\ f_y$ O.K. $s \leq \dfrac{700\cdot(1.00)}{1.2\cdot(30.08)} - 2\cdot(2.064) = 19.4 - 4.1 = 15.3$ in

$s \leq 15.3$ in

Therefore, spacing prov'd. = 6 in < 15.3 in O.K.

Use: #9 at 6" c-c spacing in span 2 (Max. positive reinforcement).

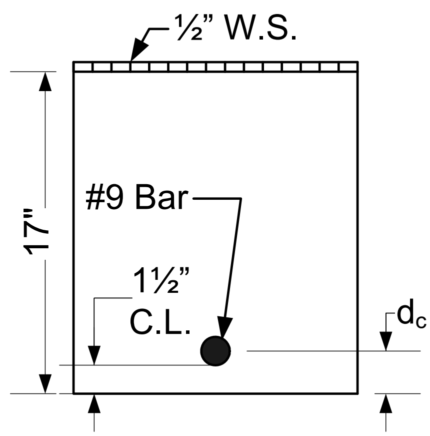


Figure E18.5

Cross Section - (0.5 pt.) Span 2

E18-1.7.3.4 Minimum Reinforcement Check

Following the procedure in E18-1.7.1.4, the minimum reinforcement check was found to be O.K.

E18-1.7.4 Negative Moment Reinforcement at Haunch/Slab Intercepts

Check the longitudinal reinforcement required at the C/L of the pier, to see if its adequate at the haunch/slab intercepts.

The haunch/slab intercepts are at (0.789 pt.) of span 1 and (0.157/0.843 pt.) of span 2. Moments at these locations are shown in Table E18.4.

Check #8 at 5" c-c spacing (as req'd. at Pier); $A_s := 1.88 \, \frac{in^2}{ft}$

Check for Strength:

Following the procedure in E18-1.7.1.1, using Strength I Limit State O.K.

Check for Fatigue:

Following the procedure in E18-1.7.1.2, using Fatigue I Limit State O.K.

Check Crack Control:

Following the procedure in E18-1.7.1.3, using Service I Limit State O.K.

Minimum Reinforcement Check:

Following the procedure in E18-1.7.1.4 O.K.

E18-1.7.5 Bar Steel Cutoffs

Select longitudinal reinforcement cutoff locations for an Interior Strip.

E18-1.7.5.1 Span 1 Positive Moment Reinforcement (Cutoffs)

Theoretical bar steel cutoff points for positive moment are determined when one-half the steel required at the (0.4 pt.) has the moment capacity, or factored resistance , M_r , equal to the total factored moment , M_u , at these points. However, the remaining bars are to be extended beyond the theoretical cutoff point and must meet the fatigue and crack control requirements at these cutoff locations. The factored moments, M_u , at the 0.1 points and haunch/slab intercepts have been plotted on Figure E18.6. The capacities, M_r , of #9 at 7" and #9 at 14" are also shown. The factored moments, M_u , and capacities, M_r , are based on Strength I Limit State criteria. The positive live load moments, M_{LL+IM} , used to calculate M_u are taken as the largest caused by live loads (LL#1 or LL#2). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM). Use maximum or minimum load factors for M_{DC} and M_{DW} (See Table E18.1) to calculate the critical force effect. When value of M_{DW} is (-), assume FWS is not present and ignore it.

Calculate the capacity of #9 at 7" c-c spacing $A_s := 1.71 \ \frac{in^2}{ft}$ $d_s := 14.9$ in

$b = 12$ inches (for a one foot design width)

As shown in E18-1.7.1.1, reinforcement will yield, therefore: $\boxed{a = 2.51}$ in

$$M_r := 0.9 \cdot (1.71) \cdot 60.0 \cdot \left(\frac{14.9 - \frac{2.51}{2}}{12} \right) \qquad \boxed{M_r = 105} \text{ kip-ft}$$

Calculate the capacity of #9 at 14" c-c spacing $A_s := 0.86 \ \frac{in^2}{ft}$ $d_s := 14.9$ in

For same section depth and less steel, reinforcement will yield, therefore: $\boxed{a = 1.26}$ in

$$M_r := 0.9 \cdot (0.86) \cdot 60.0 \cdot \left(\frac{14.9 - \frac{1.26}{2}}{12} \right) \qquad \boxed{M_r = 55.2} \text{ kip-ft}$$

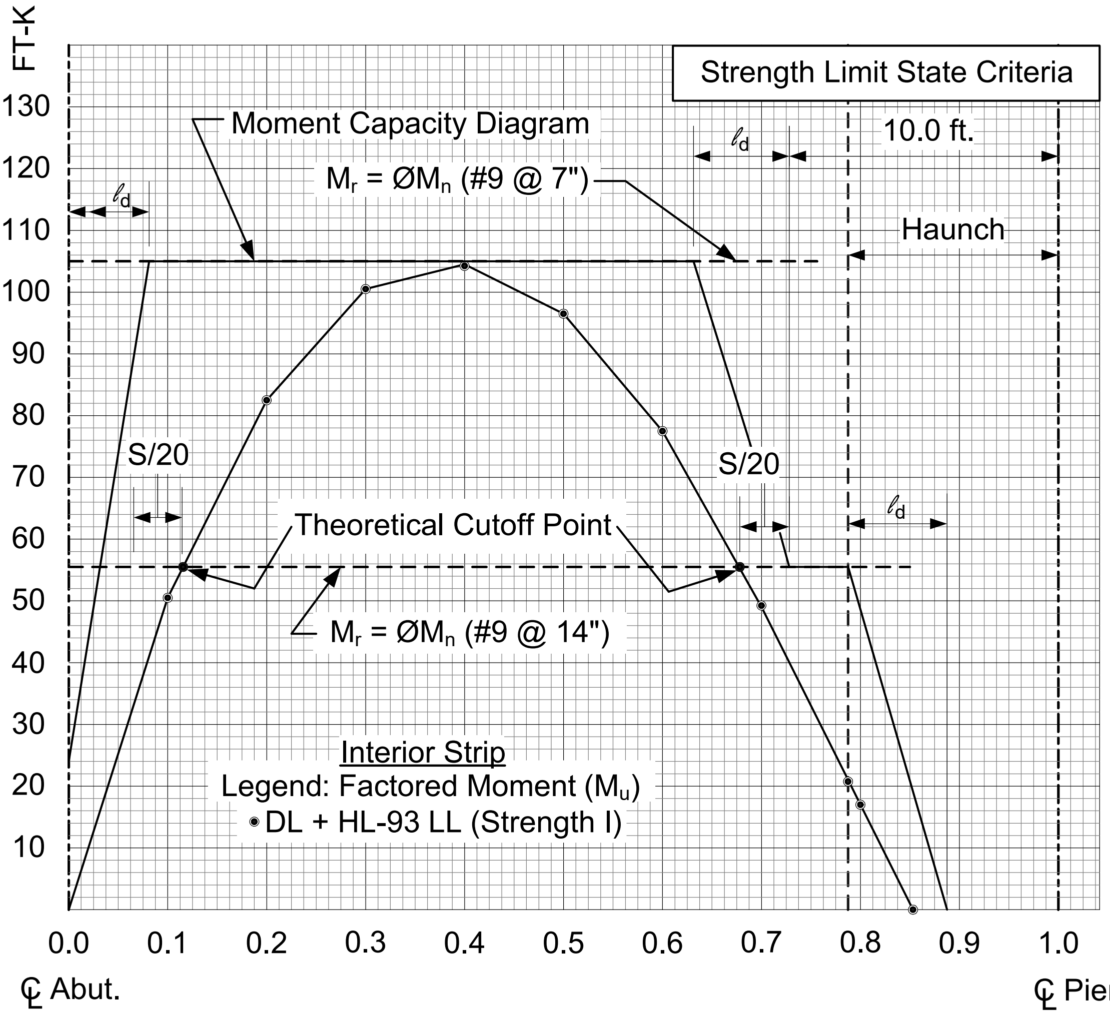


Figure E18.6

Span (1) - Positive Moment Cutoff Diagram

The moment diagram equals the capacity of #9 at 14" at 4.2 (ft) from the C/L of the abutment. Reinforcement shall be extended beyond this point a distance equal to the effective depth of the slab, 15 bar diameters, or 1/20 of the clear span, whichever is greater. **LRFD [5.10.8.1.2a]**

$d_{eff} := 14.9$ in

ℓ_d (#9) (See Table 9.9-2, Chapter 9).

$15 \cdot (d_b) = 15 \cdot (1.128) = 16.9$ in

$\frac{S}{20} = \frac{38}{20} = 1.9$ ft controls

Therefore, 1/2 of bars may be cut at 2.0 (ft) from the C/L of the abutment if fatigue and crack control criteria are satisfied.

Because the cutoff point is close to the abutment, don't cut 1/2 of bars, but run all #9 bars into the support. **LRFD [5.10.8.1.2b]**

The moment diagram equals the capacity of #9 at 14" at 12.1 (ft) from the C/L of pier. Reinforcement shall be extended S/20 beyond this point.

Therefore, 1/2 of bars may be cut at 10.0 (ft) from the C/L of pier if fatigue and crack control criteria are satisfied. (Check at 0.74 pt.)

E18-1.7.5.1.1 Fatigue Check (at Cutoff) - (0.74 Pt.)

Following the procedure in E18-1.7.1.2, using Fatigue I Limit State:

$1.75 \cdot (f_{range}) \le 26 - 0.37 \cdot f_{min}$ (for f_y = 60 ksi)

Interpolating from Table E18.4, the moments at (0.74 pt.) of span 1 are:

M_{DC} = -10.0 kip-ft M_{DW} = -0.89 kip-ft

+Fatigue Truck = 9.72 kip-ft -Fatigue Truck = -10.34 kip-ft

In regions of compressive stress due to unfactored permanent loads, fatigue shall be considered only if this compressive stress is less than ($\gamma_{LLfatigue}$ = 1.75) times the maximum tensile stress from the fatigue load. **LRFD [5.5.3.1]**

For simplicity, assume fatigue criteria should be checked.

Calculate fatigue moment: $M_{fatigue} = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.75(\text{Fatigue Truck})$

$M_{fatigueMax} := 1.0 \cdot (-10.0) + 1.0(-0.89) + 1.75(9.72)$ $\boxed{M_{fatigueMax} = 6.12}$ kip-ft (tens.)

$M_{fatigueMin} := 1.0 \cdot (-10.0) + 1.0(-0.89) + 1.75(-10.34)$ $\boxed{M_{fatigueMin} = -28.98}$ kip-ft (compr.)

Looking at values of $M_{fatigue}$ shows that the reinforcement goes through tensile and compressive stress during the fatigue cycle.

Following the procedure outlined in E18-1.7.5.2.1, fatigue criteria at bar cutoff is O.K.

E18-1.7.5.1.2 Crack Control Check (at Cutoff) - (0.74 Pt.)

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**

Following the procedure in E18-1.7.1.3, using Service I Limit State:

$\boxed{f_r = 0.48}$ ksi $\boxed{f_{r80\%} = 0.38}$ ksi $\boxed{c = 8.5}$ in $\boxed{I_g = 4913}$ in^4

$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$

Interpolating from Table E18.4, the moments at (0.74 pt.) of span 1 are:

M_{DC} = -10.0 kip-ft M_{DW} = -0.89 kip-ft M_{LL+IM} = 4.7 + 21.1 = 25.8 kip-ft (LL#1)

$M_s := 1.0\cdot(-10.0) + 1.0\cdot(25.8)$ $\boxed{M_s = 15.8}$ kip-ft

M_{DW} (FWS) moment was ignored in order to obtain a greater tensile moment

$f_T = \dfrac{M_s \cdot c}{I_g}$ $f_T := \dfrac{15.8\cdot(8.5)\cdot 12}{4913}$ $\boxed{f_T = 0.33}$ ksi

f_T =0.33 ksi < 80% f_r = 0.38 ksi; therefore, crack control criteria check not req'd.

Therefore, crack control criteria at bar cutoff is O.K.

E18-1.7.5.1.3 Minimum Reinforcement Check

Following the procedure in E18-1.7.1.4, the minimum reinforcement check was found to be O.K.

Therefore cut 1/2 of bars at 10.0 (ft) from the C/L of pier. Remaining bars are extended (ℓ_d) beyond the haunch/slab intercept as shown on Standard 18.01.

E18-1.7.5.2 Span 2 Positive Moment Reinforcement (Cutoffs)

Theoretical bar steel cutoff points for positive moment are determined when one-half the steel required at the (0.5 pt.) has the moment capacity, or factored resistance , M_r , equal to the total factored moment , M_u , at these points. However, the remaining bars are to be extended beyond the theoretical cutoff point and must meet the fatigue and crack control requirements at these cutoff locations. The factored moments, M_u , at the 0.1 points and haunch/slab intercepts have been plotted on Figure E18.7. The capacities, M_r , of #9 at 6" and #9 at 12" are also shown. The factored moments, M_u , and capacities, M_r , are based on Strength I Limit State criteria. The positive live load moments, M_{LL+IM} , used to calculate M_u are taken as the largest caused by live loads (LL#1 or LL#2). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM). Use maximum or minimum load factors for M_{DC} and M_{DW} (See Table E18.1) to calculate the critical force effect. When value of M_{DW} is (-), assume FWS is not present and ignore it.

Calculate the capacity of #9 at 6" c-c spacing $A_s := 2.00 \ \frac{in^2}{ft}$ $d_s := 14.9$ in

$b = 12$ inches (for a one foot design width)

As shown in E18-1.7.3.1, reinforcement will yield, therefore: $\boxed{a = 2.94}$ in

$$M_r := 0.9 \cdot (2.00) \cdot 60.0 \cdot \left(\frac{14.9 - \frac{2.94}{2}}{12} \right) \qquad \boxed{M_r = 120.9} \text{ kip-ft}$$

Calculate the capacity of #9 at 12" c-c spacing $A_s := 1.00 \ \frac{in^2}{ft}$ $d_s := 14.9$ in

For same section depth and less steel, reinforcement will yield, therefore: $\boxed{a = 1.47}$ in

$$M_r := 0.9 \cdot (1.00) \cdot 60.0 \cdot \left(\frac{14.9 - \frac{1.47}{2}}{12} \right) \qquad \boxed{M_r = 63.7} \text{ kip-ft}$$

The moment diagram equals the capacity of #9 at 12" at 14.4 (ft) from the C/L of pier. Reinforcement shall be extended beyond this point a distance equal to the effective depth of the slab, 15 bar diameters, or 1/20 of the clear span, whichever is greater. **LRFD [5.10.8.1.2a]**

$\frac{S}{20} = \frac{51}{20} = 2.55$ ft <u>controls</u> ℓ_d (#9) (See Table 9.9-2, Chapter 9).

Therefore, 1/2 of bars may be cut at 11.5 (ft) from the C/L of each pier if fatigue and crack control criteria are satisfied (Check at 0.23 pt.).

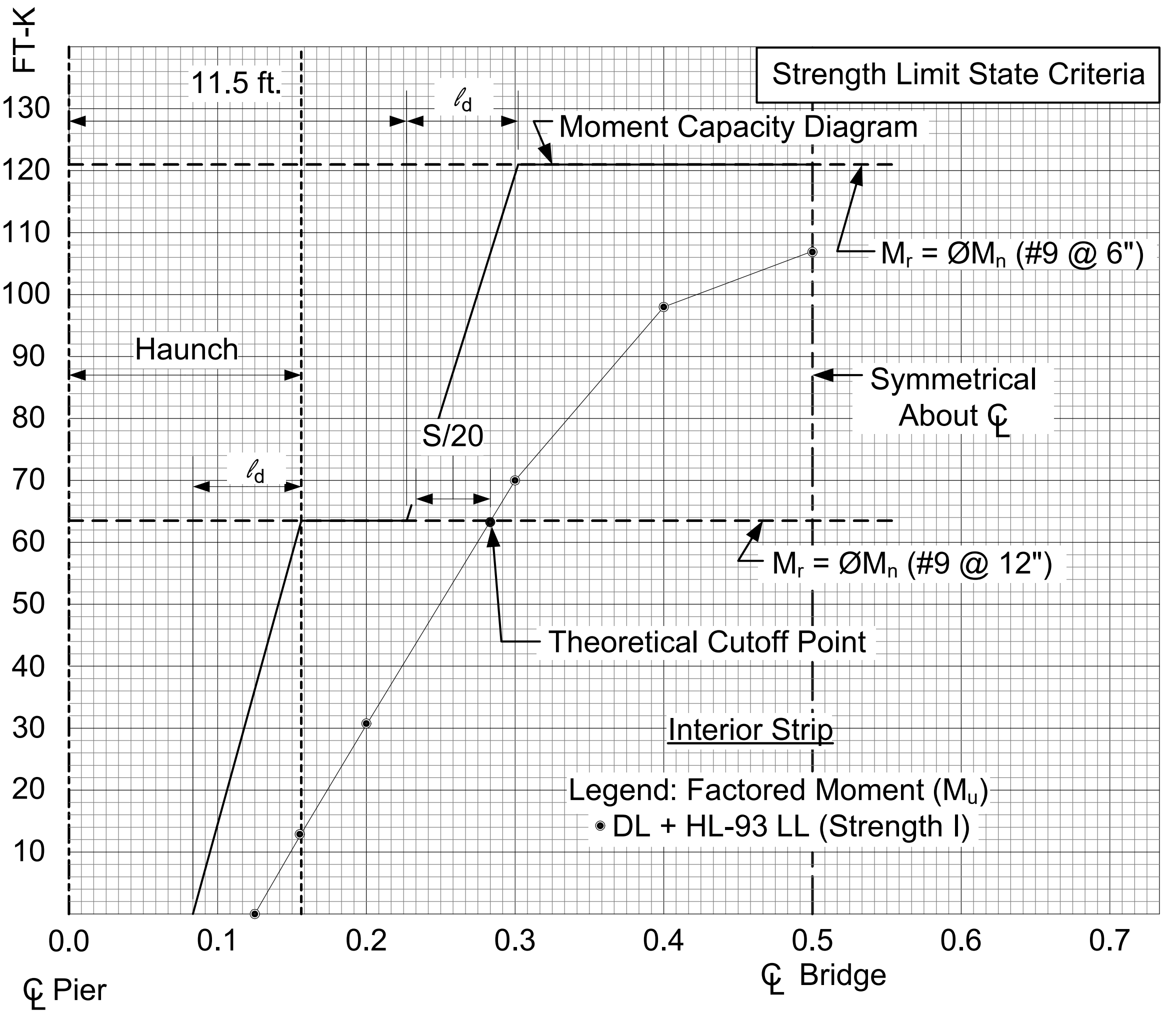


Figure E18.7

Span (2) - Positive Moment Cutoff Diagram

E18-1.7.5.2.1 Fatigue Check (at Cutoff) - (0.23 Pt.)

Looking at E18-1.2: $\eta_i := 1.0$ and from Table E18.1: $\gamma_{LLfatigue} := 1.75$ $\phi_{fatigue} := 1.0$

When reinforcement goes through tensile and compressive stress during the fatigue cycle,

$$Q = f_s + f'_s$$

Where:

f_s = tensile part of stress range in bar reinforcement due to dead load moments from applied loads in E18-1.2 and largest factored tensile moment caused by Fatigue Truck (LL#4)

f'_s = compressive part of stress range in bar reinforcement due to dead load moments from applied loads in E18-1.2 and largest factored compressive moment caused by Fatigue Truck (LL#4)

All live load moments in f_s and f'_s are multiplied by (η_i) and ($\gamma_{LLfatigue}$)

See Table E18.2 and E18.3 in E18-1.4 for description of live load and dynamic load allowance (IM).

$R_n = (\Delta F_{TH}) = 26 - 0.37 \cdot f_{min}$ (for f_y = 60 ksi) (See 18.3.5.2.1)

$R_r = \phi_{fatigue} \cdot R_n = 1.0 \cdot (26 - 0.37 \cdot f_{min})$

Therefore: $f_s + f'_s \le 26 - 0.37 \cdot f_{min}$ (Limit States Equation)

Interpolating from Table E2, the moments at (0.23 pt.) of span 2 are:

M_{DC} = -3.5 kip-ft M_{DW} = -0.31 kip-ft

+Fatigue Truck = 10.02 kip-ft -Fatigue Truck = -7.3 kip-ft

In regions of compressive stress due to unfactored permanent loads, fatigue shall be considered only if this compressive stress is less than ($\gamma_{LLfatigue}$ = 1.75) times the maximum tensile stress from the fatigue load. **LRFD [5.5.3.1]**

The section properties for fatigue shall be based on cracked sections where the sum of stresses, due to unfactored permanent loads, and ($\gamma_{LLfatigue}$ = 1.75) times the fatigue load is tensile and exceeds $0.095\sqrt{f'_c}$.

For simplicity, assume fatigue criteria should be checked and use cracked section properties.

Calculate fatigue moment: $M_{fatigue}$ = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.75(Fatigue Truck)

$M_{fatigueMax} := 1.0(-3.5) + 1.75(10.02)$ $\boxed{M_{fatigueMax} = 14.04}$ kip-ft (tension)

$M_{fatigueMin} := 1.0(-3.5) + 1.75(-7.3)$ $\boxed{M_{fatigueMin} = -16.27}$ kip-ft (compression)

M_{DW} (FWS) moment was ignored in order to obtain a greater tensile range.

Looking at values of $M_{fatigue}$, shows that the reinforcement goes through tensile and compressive stress during the fatigue cycle

See Figure E18.8, for definition of d_1, d_2, d', A_s and A'_s.

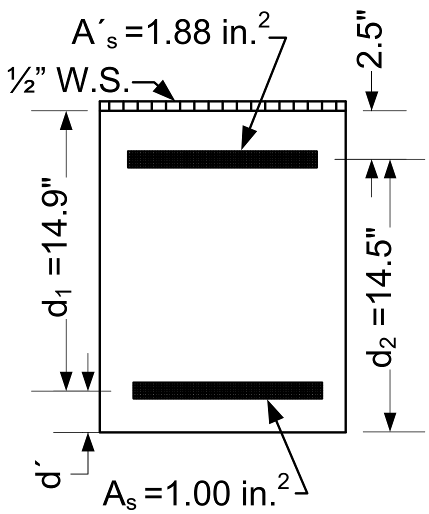


Figure E18.8

Cross Section - (0.23 pt.) Span 2

The moment arm used in equations below is: $(j_1)\ (d_1)$ for finding f_s
$(j_2)\ (d_2)$ for finding f'_s

Using: A_s = 1.00 in²/ft, d_1 = 14.9 in, n = 8, and transformed section analysis, gives a value of $j_1 = 0.914$

Using: A'_s = 1.88 in²/ft, d_2 = 14.5 in, n = 8, and transformed section analysis, gives a value of $j_2 = 0.887$; $k = x/d_2 = 0.34$, where x = distance from compression face to neutral axis

The tensile part of the stress range in the bottom bars is computed as:

$$f_s := \frac{M_{fatigueMax} \cdot 12}{A_s \cdot (j_1) \cdot d_1} \qquad \boxed{f_s = 12.37} \text{ ksi (tension)}$$

The compressive part of the stress range in the bottom bars is computed as:

$$f'_s := \frac{M_{fatigueMin} \cdot 12}{A'_s \cdot (j_2) \cdot d_2} \cdot \frac{k - \left(\frac{d'}{d_2}\right)}{1 - k} \qquad \boxed{f'_s = -2.42} \text{ ksi (compression)}$$

It is assumed (#8's at 5") req'd. at pier, is present at this location as compression steel (A'_s).

Therefore, total stress range on bottom steel:

$f_s + f'_s = 12.37 - (-2.42) = 14.79$ ksi

$R_r := 26 - 0.37 \cdot f_{min}$ where $f_{min} = f'_s$, therefore: $\boxed{R_r = 26.9}$ ksi

Therefore, $f_s + f'_s = 14.79$ ksi < $R_r = 26.9$ ksi <u>O.K..</u>

E18-1.7.5.2.2 Crack Control Check (at Cutoff) - (0.23 Pt.)

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**.

Following the procedure in E18-1.7.1.3, using Service I Limit State:

$\boxed{f_r = 0.48}$ ksi $\boxed{f_{r80\%} = 0.38}$ ksi $\boxed{c = 8.5}$ in $\boxed{I_g = 4913}$ in^4

$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$

Interpolating from Table E18.4, the moments at (0.23 pt.) of span 2 are:

$M_{DC} = -3.51$ kip-ft $M_{DW} = -0.31$ kip-ft $M_{LL+IM} = 3.7 + (21.9) = 25.6$ kip-ft (LL#1)

$M_s := 1.0 \cdot (-3.51) + 1.0 \cdot (25.6)$ $\boxed{M_s = 22.1}$ kip-ft

M_{DW} (FWS) moment was ignored in order to obtain a greater tensile moment.

$f_T = \frac{M_s \cdot c}{I_g}$ $f_T := \frac{22.1 \cdot (8.5) \cdot 12}{4913}$ $\boxed{f_T = 0.46}$ ksi

$f_T = 0.46$ ksi > 80% $f_r = 0.38$ ksi; therefore, check crack control criteria

For: #9 at 12" c-c spacing ($A_s = 1.00\ in^2/ft$)

The values for γ_e, d_c, h, and β_s, used to calculate max. spacing (s) of reinforcement are :

$\gamma_e := 1.00$ for Class 1 exposure condition (bottom reinforcement)

$\boxed{d_c = 2.064}$ in $\boxed{h = 17}$ in $\boxed{\beta_s = 1.2}$

f_{ss} = tensile stress in steel reinforcement at the Service I Limit State (ksi) $\leq 0.6\ f_y$

The moment arm used to calculate f_{ss} is: (j) (h - d_c)

As shown in fatigue calculations in E18-1.7.5.2.1, j = 0.914

The value of f_{ss} and (s) are:

$\boxed{f_{ss} = 19.43}$ ksi $\leq 0.6\, f_y$ O.K. $\quad s \leq \dfrac{700 \cdot (1.00)}{1.2 \cdot (19.43)} - 2 \cdot (2.064) = 30.07 - 4.1 = 26.0$ in

$s \leq 26.0$ in

Therefore, spacing prov'd. = 12 in < 26.0 in O.K.

E18-1.7.5.2.3 Minimum Reinforcement Check

Following the procedure in E18-1.7.1.4, the minimum reinforcement check was found to be O.K.

Therefore, cut 1/2 of bars at 11.5 (ft) from the C/L of each pier. Remaining bars are extended (ℓ_d) beyond the haunch/slab intercept as shown on Standard 18.01.

E18-1.7.5.3 Span 1 Negative Moment Reinforcement (Cutoffs)

Theoretical bar steel cutoff points for negative moment are determined when one-half the steel required at the (C/L Pier) has the moment capacity, or factored resistance , M_r , equal to the total factored moment , M_u , at these points. However, the remaining bars are to be extended beyond the theoretical cutoff point and must meet the fatigue and crack control requirements at these cutoff locations. The factored moments, M_u , at the 0.1 points and haunch/slab intercepts have been plotted on Figure E18.9. The capacities, M_r , of #8 at 5" and #8 at 10" are also shown. The factored moments, M_u , and capacities, M_r , are based on Strength I Limit State criteria. The negative live load moments, M_{LL+IM} , used to calculate M_u are taken as the largest caused by live loads (LL#1, LL#2 or LL#3). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM). Use maximum or minimum load factors for M_{DC} and M_{DW} (See Table E18.1) to calculate the critical force effect. When value of M_{DW} is (+), assume FWS is not present and ignore it.

Calculate the capacity of #8 at 5" c-c spacing $\quad A_s := 1.88\ \dfrac{in^2}{ft}$

$b = 12$ inches (for a one foot design width)

As shown in E18-1.7.2.1, reinforcement will yield, therefore: $\boxed{a = 2.76}$ in

$M_r = 204.1$ kip-ft (at C/L pier), $\quad d_s := 25.5$ in

$M_r = 111.0$ kip-ft (in span), $\quad d_s := 14.5$ in

Calculate the capacity of #8 at 10" c-c spacing $\quad A_s := 0.94\ \dfrac{in^2}{ft}$

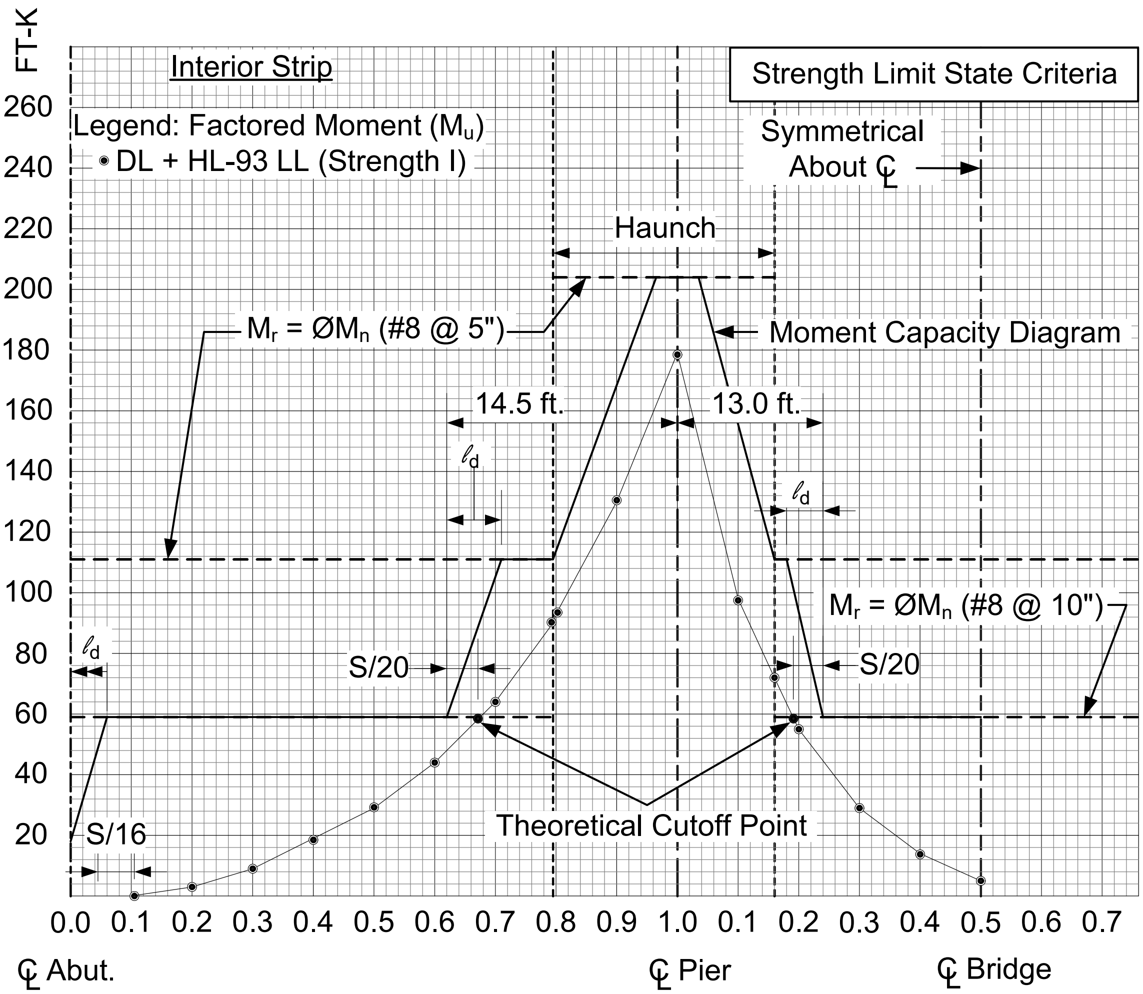


Figure E18.9

Negative Moment Cutoff Diagram

For same section depth and less steel, reinforcement will yield, therefore:

$\boxed{a = 1.38}$ in

M_r = 104.9 kip-ft (at C/L pier), $d_s := 25.5$ in

M_r = 58.4 kip-ft (in span), $d_s := 14.5$ in

The moment diagram equals the capacity of #8 at 10" at 12.5 (ft) from the C/L of pier. Reinforcement shall be extended beyond this point a distance equal to the effective depth of the slab, 15 bar diameters, or 1/20 of the clear span, whichever is greater. **LRFD [5.10.8.1.2a]**

$\frac{S}{20} = \frac{38}{20} = 1.9$ ft <u>controls</u> ℓ_d (#8) (See Table 9.9-2, Chapter 9)

Therefore, 1/2 of bars may be cut at 14.5 (ft) from the C/L of pier if fatigue and crack control criteria are satisfied. (Check at 0.62 pt.)

E18-1.7.5.3.1 Fatigue Check (at Cutoff) - (0.62 Pt.)

Following the procedure in E18-1.7.5.2.1, using Fatigue I Limit State:

$f_s + f'_s \le 26 - 0.37 \cdot f_{min}$ (for f_y = 60 ksi)

Interpolating from Table E18.4, the moments at (0.62 pt.) of span 1 are:

M_{DC} = 4.44 kip-ft M_{DW} = 0.4 kip-ft

+Fatigue Truck = 13.7 kip-ft -Fatigue Truck = -8.68 kip-ft

For simplicity, assume fatigue criteria should be checked and use cracked section properties.

Calculate fatigue moment: $M_{fatigue} = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.75(\text{Fatigue Truck})$

$M_{fatigueMax} := 1.0(4.44) + 1.75(-8.68)$ $\boxed{M_{fatigueMax} = -10.75}$ kip-ft (tension)

$M_{fatigueMin} := 1.0(4.44) + 1.75(13.7)$ $\boxed{M_{fatigueMin} = 28.41}$ kip-ft (compression)

M_{DW} (FWS) moment was ignored in order to obtain a greater tensile range.

Looking at values of $M_{fatigue}$, shows that the reinforcement goes through tensile and compressive stress during the fatigue cycle

See Figure E18.10, for definition of d_1, d_2, d', A_s and A'_s.

The moment arm used in equations below is: $(j_1)\ (d_1)$ for finding f_s
$(j_2)\ (d_2)$ for finding f'_s

Using: A_s = 0.94 in^2/ft, d_1 = 14.5 in, n = 8, and transformed section analysis, gives a value of j_1 = 0.915

Using: A'_s =1.71 in^2/ft, d_2 = 14.9 in, n = 8, and transformed section analysis, gives a value of j_2 = 0.893; k = x/d_2 = 0.33, where x = distance from compression face to neutral axis

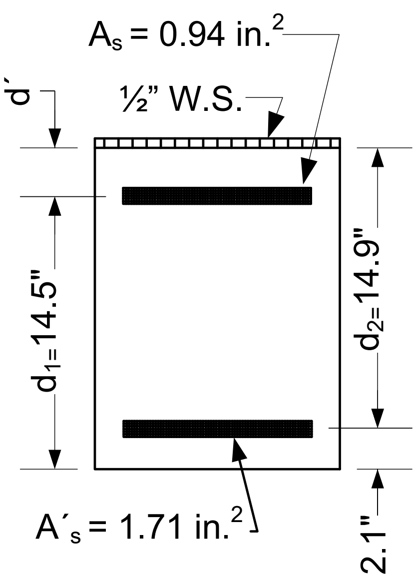


Figure E18.10

Cross Section - (0.62 pt.) Span 1

The tensile part of the stress range in the top bars is computed as:

$$f_s := \frac{M_{fatigueMax} \cdot 12}{A_s \cdot (j_1) \cdot d_1} \qquad \boxed{f_s = 10.34} \text{ ksi} \quad \text{(tension)}$$

The compressive part of the stress range in the top bars is computed as:

$$f'_s := \frac{M_{fatigueMin} \cdot 12}{A'_s \cdot (j_2) \cdot d_2} \cdot \frac{k - \left(\frac{d'}{d_2}\right)}{1 - k} \qquad \boxed{f'_s = -3.63} \text{ ksi} \quad \text{(compression)}$$

It is assumed (#9's at 7") is present at this location as compression steel (A'_s).

Therefore, total stress range on top steel:

$$f_s + f'_s = 10.34 - (-3.63) = 13.97 \text{ ksi}$$

$R_r := 26 - 0.37 \cdot f_{min}$ where f_{min} = f'_s , therefore: $\boxed{R_r = 27.34}$ ksi

Therefore, $f_s + f'_s$ = 13.97 ksi < R_r = 27.34 ksi O.K..

E18-1.7.5.3.2 Crack Control Check (at Cutoff) - (0.62 Pt.)

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**.

Following the procedure in E18-1.7.1.3, using Service I Limit State:

$\boxed{f_r = 0.48}$ ksi $\boxed{f_{r80\%} = 0.38}$ ksi $\boxed{c = 8.5}$ in $\boxed{I_g = 4913}$ in^4

$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$

Interpolating from Table E18.4, the moments at (0.62 pt.) of span 1 are:

$M_{DC} = 4.4$ kip-ft $M_{DW} = 0.4$ kip-ft $M_{LL+IM} = -5.88 + (-23.88) = -29.8$ kip-ft (LL#2)

$M_s := 1.0 \cdot (4.4) + 1.0 \cdot (-29.8)$ $\boxed{M_s = -25.4}$ kip-ft

M_{DW} (FWS) moment was ignored in order to obtain a greater tensile moment.

$f_T = \dfrac{M_s \cdot c}{I_g}$ $f_T := \dfrac{25.4 \cdot (8.5) \cdot 12}{4913}$ $\boxed{f_T = 0.53}$ ksi

$f_T = 0.53$ ksi > 80% $f_r = 0.38$ ksi; therefore, check crack control criteria

For: #8 at 10" c-c spacing ($A_s = 0.94\ in^2/ft$)

The values for γ_e , d_c , h , and β_s , used to calculate max. spacing (s) of reinforcement are :

$\gamma_e := 0.75$ for Class 2 exposure condition (top reinforcement)

$\boxed{d_c = 2.5}$ in $\boxed{h = 17}$ in $\boxed{\beta_s = 1.25}$

f_{ss} = tensile stress in steel reinforcement at the Service I Limit State (ksi) $\leq 0.6\ f_y$

The moment arm used to calculate f_{ss} is: (j) (h - d_c)
As shown in fatigue calculations in E18-1.7.5.3.1, j = 0.915

The value of f_{ss} and (s) are:

$\boxed{f_{ss} = 24.44}$ ksi < 0.6 f_y O.K. $s \leq \dfrac{700 \cdot (0.75)}{1.25 \cdot (24.44)} - 2 \cdot (2.50) = 17.2 - 5.0 = 12.2$ in

$s \leq 12.2$ in

Therefore, spacing prov'd. = 10 in < 12.2 in <u>O.K.</u>

E18-1.7.5.3.3 Minimum Reinforcement Check

Following the procedure in E18-1.7.2.4, the minimum reinforcement check was found to be O.K.

Therefore, cut 1/2 of bars at 14.5 (ft) from the C/L of the pier. Remaining bars are extended beyond the point of inflection, a distance equal to the effective depth of the slab, 12 bar diameters, or 1/16 of the clear span, whichever is greater. **LRFD [5.10.8.1.2c]**

$d_{eff} := 14.5$ in $\qquad\qquad \ell_d$ (#8) (See Table 9.9-2, Chapter 9)

$12 \cdot (d_b) = 12 \cdot (1.00) = 12.0$ in

$\frac{S}{16} = \frac{38}{16} = 2.38$ ft $\qquad$ controls

Looking at the factored moment diagram (M_u) on Figure E18.9, the point of inflection is found at the (0.11 pt.). Therefore, the remaining bars could be terminated at 36.5 (ft) from the C/L of pier and these bars lapped with smaller size bars spaced at 10 inches.

Because this bar termination point is close to the abutment, run remaining bars (#8 at 10" c-c spacing) to the end of the slab.

E18-1.7.5.4 Span 2 Negative Moment Reinforcement (Cutoffs)

Capacities of #8 at 5" and #8 at 10" c-c spacing are stated in E18-1.7.5.3

The moment diagram equals the capacity of #8 at 10 " at 10.0 (ft) from the C/L of pier. Reinforcement shall be extended beyond this point a distance equal to the effective depth of the slab, 15 bar diameters, or 1/20 of the clear span, whichever is greater. **LRFD [5.10.8.1.2a]**

$\frac{S}{20} = \frac{51}{20} = 2.55$ ft $\qquad$ controls $\qquad \ell_d$ (#8) (See Table 9.9-2, Chapter 9)

Therefore, 1/2 of bars may be cut at 13.0 (ft) from the C/L of pier if fatigue and crack control criteria are satisfied. (Check at 0.25 pt.)

E18-1.7.5.4.1 Fatigue Check (at Cutoff) - (0.25 Pt.)

Following the procedure in E18-1.7.5.2.1, using Fatigue I Limit State:

$f_s + f'_s \le 26 - 0.37 \cdot f_{min}$ $\qquad$ (for f_y = 60 ksi)

Interpolating from Table E18.4, the moments at (0.25 pt.) of span 2 are:

M_{DC} = -0.45 kip-ft $\qquad$ M_{DW} = -0.05 kip-ft

+Fatigue Truck = 10.9 kip-ft $\qquad$ -Fatigue Truck = -7.0 kip-ft

For simplicity, assume fatigue criteria should be checked and use cracked section properties.

Calculate fatigue moment: $M_{fatigue} = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.75(\text{Fatigue Truck})$

$M_{fatigueMax} := 1.0(-0.45) + 1.0(-0.05) + 1.75(-7.0)$ $\boxed{M_{fatigueMax} = -12.8}$ kip-ft (tension)

$M_{fatigueMin} := 1.0(-0.45) + 1.0(-0.05) + 1.75(10.9)$ $\boxed{M_{fatigueMin} = 18.57}$ kip-ft (compr.)

Looking at values of $M_{fatigue}$, shows that the reinforcement goes through tensile and compressive stress during the fatigue cycle

See Figure E18.11, for definition of d_1, d_2, d', A_s and A'_s.

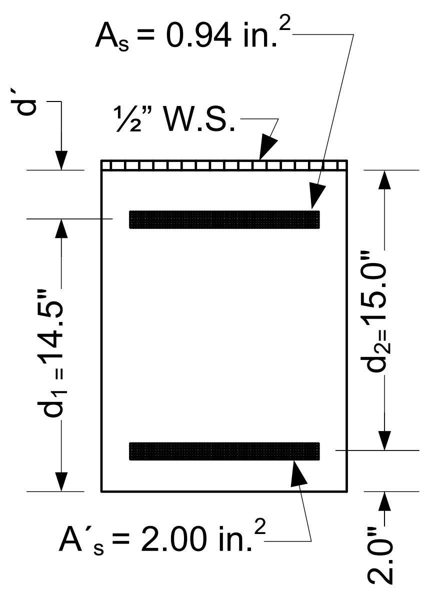


Figure E18.11

Cross Section - (0.25 pt.) Span 2

The moment arm used in equations below is: $(j_1)\ (d_1)$ for finding f_s
$(j_2)\ (d_2)$ for finding f'_s

Using: A_s = 0.94 in^2/ft, d_1 = 14.5 in, n = 8, and transformed section analysis, gives a value of j_1 = 0.915

Using: A'_s =2.00 in^2/ft, d_2 = 15.0 in, n = 8, and transformed section analysis, gives a value of j_2 = 0.886; $k = x/d_2$ = 0.34, where x = distance from compression face to neutral axis

The tensile part of the stress range in the top bars is computed as:

$$f_s := \frac{M_{fatigueMax} \cdot 12}{A_s \cdot (j_1) \cdot d_1}$$ $\boxed{f_s = 12.27}$ ksi (tension)

The compressive part of the stress range in the top bars is computed as:

$$f'_s := \frac{M_{fatigueMin} \cdot 12}{A'_s \cdot (j_2) \cdot d_2} \cdot \frac{k - \left(\frac{d'}{d_2}\right)}{1 - k}$$ $\boxed{f'_s = -2.2}$ ksi (compression)

It is assumed (#9's at 6") is present at this location as compression steel (A'_s).

Therefore, total stress range on top steel:

$f_s + f'_s = 12.27 - (-2.20) = 14.47$ ksi

$R_r := 26 - 0.37 \cdot f_{min}$ where $f_{min} = f'_s$, therefore: $\boxed{R_r = 26.81}$ ksi

Therefore, $f_s + f'_s = 14.47$ ksi < $R_r = 26.81$ ksi <u>O.K..</u>

E18-1.7.5.4.2 Crack Control Check (at Cutoff) - (0.25 Pt.)

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**.

Following the procedure in E18-1.7.1.3, using Service I Limit State:

$\boxed{f_r = 0.48}$ ksi $\boxed{f_{r80\%} = 0.38}$ ksi $\boxed{c = 8.5}$ in $\boxed{I_g = 4913}$ in^4

$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$

Interpolating from Table E18.4, the moments at (0.25 pt.) of span 2 are:

M_{DC} = -0.45 kip-ft M_{DW} = -0.05 kip-ft M_{LL+IM} = -4.35 + (-18.25) = -22.6 kip-ft (LL#2)

$M_s := 1.0 \cdot (0.45) + 1.0(0.05) + 1.0 \cdot (22.6)$ $\boxed{M_s = 23.1}$ kip-ft

$f_T = \frac{M_s \cdot c}{I_g}$ $f_T := \frac{23.1 \cdot (8.5) \cdot 12}{4913}$ $\boxed{f_T = 0.48}$ ksi

f_T =0.48 ksi > 80% f_r = 0.38 ksi; therefore, check crack control criteria

For: #8 at 10" c-c spacing ($A_s = 0.94\ in^2/ft$)

The values for γ_e, d_c, h, and β_s, used to calculate max. spacing (s) of reinforcement are :

$\gamma_e := 0.75$ for Class 2 exposure condition (top reinforcement)

$\boxed{d_c = 2.5}$ in $\boxed{h = 17}$ in $\boxed{\beta_s = 1.25}$

f_{ss} = tensile stress in steel reinforcement at the Service I Limit State (ksi) $\leq$ 0.6 f_y

The moment arm used to calculate f_{ss} is: (j) (h - d_c)
As shown in fatigue calculations in E18-1.7.5.3.1, j = 0.915

The value of f_{ss} and (s) are:

$\boxed{f_{ss} = 22.23}$ ksi < 0.6 f_y O.K. $s \leq \frac{700 \cdot (0.75)}{1.25 \cdot (22.23)} - 2 \cdot (2.50) = 19.0 - 5.0 = 14.0$ in

$s \leq 14.0$ in

Therefore, spacing prov'd. = 10 in < 14.0 in O.K.

E18-1.7.5.4.3 Minimum Reinforcement Check

Following the procedure in E18-1.7.2.4, the minimum reinforcement check was found to be O.K.

Therefore, cut 1/2 of bars at 13.0 (ft) from the C/L of the pier. Remaining bars are extended beyond the point of inflection, a distance equal to the effective depth of the slab, 12 bar diameters, or 1/16 of the clear span, whichever is greater. **LRFD [5.10.8.1.2c]**

$\frac{S}{16} = \frac{51}{16} = 3.19$ ft controls ℓ_d (#8) (See Table 9.9-2, Chapter 9)

Looking at the factored moment diagram (M_u) on Figure E18.9, no point of inflection is found in span 2.

Therefore, run the remaining bars (#8 at 10" c-c spacing) to the C/L of span 2 and lap them.

E18-1.8 Evaluation of Longitudinal Reinforcement for Permit Vehicle

Check the adequacy of the longitudinal reinforcement to see if it has the moment capacity to handle factored moments due to applied dead load (including future wearing surface) and the Wisconsin Standard Permit Vehicle (Wis-SPV) (with a minimum gross vehicle load of 190 kips) on an interior strip. This requirement is stated in 17.1.2.1.

The Wisconsin Standard Permit Vehicle load that can be carried by the bridge is 225 kips, when the future wearing surface is present. Details for the calculation of this load are shown in Chapter 45, "Reinforced Concrete Slab Rating" example.

Wisconsin Standard Permit Vehicle (Wis-SPV) load capacity = 225 kips > 190 kips O.K.

E18-1.9 Longitudinal Reinforcement in Bottom of Haunch

At least (1/4) of maximum positive moment reinforcement in continuous-spans shall extend into the support **LRFD [5.10.8.1.2b]**.

Max. positive (A_s) = $2.00 \ \frac{in^2}{ft}$ (#9 at 6" c-c spacing, in span 2)

Reinf. req'd. = $0.25 \cdot (2.00) = 0.5 \ \frac{in^2}{ft}$

Therefore, use #7 at 13 in. (0.55 in^2/ft) > reinf. req'd and min. reinf. on Standard 18.01 O.K.

See Figure E18.12 for a summary of longitudinal reinforcement selected and layout of transverse distribution steel selected in E18-1.12.

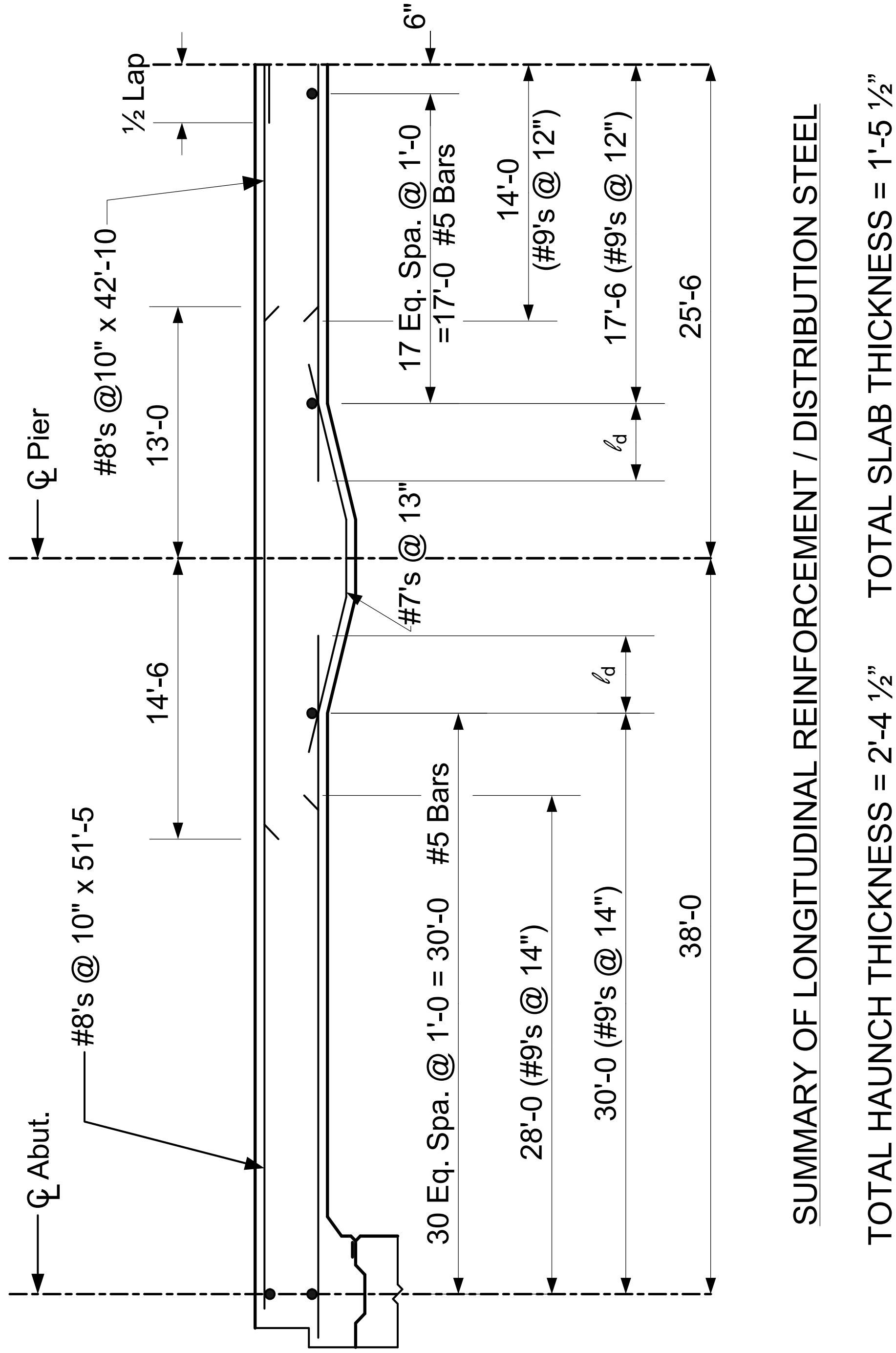


Figure E18.12

Summary of Longitudinal Reinforcement / Distribution Steel

E18-1.10 Live Load Distribution (Exterior Strip)

The exterior strip width (E), is assumed to carry one wheel line and a tributary portion of design lane load **LRFD [4.6.2.1.4]**.

(E) equals the distance between the edge of the slab and the inside face of the barrier, plus 12 inches, plus 1/4 of the full strip width specified in **LRFD [4.6.2.3]**.

The exterior strip width (E) shall not exceed either 1/2 the full strip width or 72 inches.

The distance from the edge of slab to the inside face of barrier = 15 inches

E18-1.10.1 Strength and Service Limit State

Use the smaller equivalent widths, which are from multi-lane loading, for full strip width when (HL-93) live load is to be distributed, for Strength I Limit State and Service I Limit State.

From previous calculations in E18-1.6:

Full strip width = 141 in. (Span 1,3) - multi-lane loading

Full strip width = 151 in. (Span 2) - multi-lane loading

The multiple presence factor (m) has been included in the equations for full strip width and therefore aren't used to adjust the distribution factor. **LRFD [3.6.1.1.2]**

Span 1, 3: $E = 15 + 12 + \frac{141}{4} = 62.2$ in.; but not to exceed ($\frac{141}{2}$ in. or 72 in.)

Therefore, E = 62.2 in. (Spans 1, 3)

Span 2: $E = 15 + 12 + \frac{151}{4} = 64.7$ in.; but not to exceed ($\frac{151}{2}$ in. or 72 in.)

Therefore, E = 64.7 in. (Span 2)

The distribution factor (DF) is computed for a design slab width equal to one foot.

Compute the distribution factor associated with one truck wheel line, to be applied to axle loads

$$DF = \frac{1wheel\_line}{\left(\frac{2wheel\_lines}{lane}\right) \cdot E}$$ (where E is in feet)

For Spans 1 & 3: (E = 62.2" = 5.183')

$$DF := \frac{1}{2 \cdot (5.183)}$$

$$\boxed{DF = 0.096} \ \frac{lanes}{ft - slab}$$

For Span 2: (E = 64.7" = 5.392')

$$DF := \frac{1}{2\cdot(5.392)} \qquad \boxed{DF = 0.093} \; \frac{lanes}{ft - slab}$$

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]**

$$DF = \left(\frac{\frac{SWL}{10ft\_lane\_load\_width}}{E} \right) \quad \text{(where E is in feet)}$$

SWL = slab width loaded = (E) - (distance from the edge of slab to inside face of barrier) (ft)

= 62.2 - 15 = 47.2 in. = 3.93 ft. (Span 1 & 3)

= 64.7 - 15 = 49.7 in. = 4.14 ft. (Span 2)

For Spans 1 & 3: (E = 5.183' ; SWL = 3.93')

$$DF := \frac{3.93 \div 10}{5.183} \qquad \boxed{DF = 0.076} \; \frac{lanes}{ft - slab}$$

For Span 2: (E = 5.392' ; SWL = 4.14')

$$DF := \frac{4.14 \div 10}{5.392} \qquad \boxed{DF = 0.077} \; \frac{lanes}{ft - slab}$$

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.

Therefore, use : <u>DF = 0.096 lanes/ft.-slab</u>, for Design Truck and Design Tandem Loads

<u>DF = 0.077 lanes/ft.-slab</u>, for Design Lane Load

The concrete parapet is not to be considered to provide strength to the exterior strip (edge beam) **LRFD [9.5.1]**.

TABLE E18.5 Unfactored Moments (kip - ft) (on a one foot design width) **Exterior Strip**

Point	M_{DC} [1]	M_{DW} [2]	DF=0.077 (IM not used) +Design Lane	DF=0.077 (IM not used) -Design Lane	DF=0.096 (incl. IM =33%) +Design Tandem	DF=0.096 (incl. IM =33%) -Design Tandem
0.1	11.9	0.8	2.9	-0.9	19.4	-3.6
0.2	19.6	1.3	5.0	-1.7	32.7	-7.2
0.3	23.0	1.6	6.4	-2.6	40.0	-10.8
0.4	22.2	1.5	7.1	-3.4	42.3	-14.4
0.5	17.3	1.2	7.1	-4.3	40.8	-18.0
0.6	8.1	0.6	6.5	-5.2	36.0	-21.7
0.7	-5.3	-0.4	5.1	-6.0	27.9	-25.2
0.789	-21.1	-1.5	3.3	-6.9	19.0	-28.3
0.8	-22.9	-1.6	3.2	-7.1	17.8	-28.8
0.9	-45.0	-3.1	2.2	-9.8	9.5	-32.4
1.0	-72.6	-4.9	2.0	-14.0	10.4	-36.0
1.1	-36.7	-2.5	1.7	-8.0	8.6	-24.6
1.157	-20.8	-1.4	2.1	-5.6	15.6	-22.3
1.2	-10.1	-0.7	2.6	-4.4	21.3	-20.8
1.3	8.8	0.6	4.9	-3.4	32.6	-16.8
1.4	20.2	1.4	6.8	-3.4	39.9	-12.9
1.5	24.0	1.6	7.4	-3.4	42.2	-9.0

Point	DF=0.096 (incl. IM =33%) +Design Truck	DF=0.096 (incl. IM =33%) -Design Truck	DF=0.077 [3] (IM not used) (90%) of -Design Lane	DF=0.096 [3] (incl. IM =33%) (90%) of -Double Design Trucks
0.1	20.4	-4.4	------	------
0.2	33.1	-8.7	------	------
0.3	38.8	-13.1	------	------
0.4	39.9	-17.4	------	------
0.5	38.2	-21.8	------	------
0.6	34.6	-26.0	------	------
0.7	26.3	-30.5	-5.4	-27.4
0.789	15.8	-34.4	-6.2	-30.9
0.8	14.7	-34.9	-6.4	-31.4
0.9	10.2	-39.1	-8.8	-35.4
1.0	11.4	-45.0	-12.6	-39.5
1.1	9.0	-26.8	-7.2	-25.5
1.157	13.6	-24.5	-5.0	-22.8
1.2	17.3	-22.7	-4.0	-20.9
1.3	31.2	-18.5	------	------
1.4	39.9	-14.1	------	------
1.5	42.0	-9.9	------	------

Superscripts for Table E18.5 are defined on the following page.

In Table E18.5:

1 M_{DC} is moment due to slab dead load (DC_{slab}), parapet dead load (DC_{para}) after its weight is distributed across exterior strip width (E) and 1/2 inch wearing surface ($DC_{1/2"WS}$).

Using average of exterior strip widths: $\frac{62.2 + 64.7}{2}$ = 63.5 in = 5.3 ft

DC_{para} = (Parapet wgt.) / 5.3 ft = (387 plf) / 5.3 ft = 73 plf (on a 1'-0 slab width)

2 M_{DW} is moment due to future wearing surface (DW_{FWS})

3 The points of contraflexure are located at the (0.66 pt.) of span 1 and the (0.25 pt.) of span 2, when a uniform load is placed across the entire structure. Negative moments in these columns are shown between the points of contraflexure per **LRFD [3.6.1.3.1]**.

E18-1.11 Longitudinal Slab Reinforcement (Exterior Strip)

Select longitudinal reinforcement for an Exterior Strip (edge beam) **LRFD [5.12.2.1]**.

The reinforcement in the Exterior Strip is always equal to or greater than that required for the slab in an Interior Strip.

The concrete cover on the top bars is 2 1/2 inches, which includes a 1/2 inch wearing surface. The bottom bar cover is 1 1/2 inches. (See 18.4.6)

E18-1.11.1 Positive Moment Reinforcement for Span 1

Examine the 0.4 point of span 1

E18-1.11.1.1 Design for Strength

Following the procedure in E18-1.7.1.1, using Strength I Limit State:

$$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM}) \leq 0.90\, A_s\, f_s\, (d_s - a/2)$$

The positive live load moment shall be the largest caused by live loads (LL#1 or LL#2). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM).

From Table E18.5, the largest live load moment is from (LL#1), therefore at (0.4 pt.) of span 1:

M_{DC} = 22.2 kip-ft $\quad$ M_{DW} = 1.5 kip-ft $\quad$ M_{LL+IM} = 7.1 + 42.3 = 49.4 kip-ft

$M_u := 1.25\cdot(22.2) + 1.50\cdot(1.5) + 1.75\cdot(49.4)$ $\quad$ $\boxed{M_u = 116.5}$ kip-ft

$b := 12$ inches (for a one foot design width) and $\boxed{d_s = 14.9}$ in

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_s, are:

$\boxed{R_u = 583}$ psi $\boxed{\rho = 0.0108}$ $\boxed{A_s = 1.93}$ $\frac{in^2}{ft}$

For Span 1 & 3:

A_s (req'd) = 1.93 $\frac{in^2}{ft}$ (to satisfy Exterior Strip requirements)

A_s (prov'd) = 1.71 $\frac{in^2}{ft}$ (#9 at 7" c-c spacing) (to satisfy Interior Strip requirements)

Therefore, <u>use: #9 at 6" c-c spacing</u> (A_s = 2.00 $\frac{in^2}{ft}$) in Exterior Strip width of 5.3 ft.

E18-1.11.1.2 Check Crack Control

Following the procedure in E18-1.7.1.3, the crack control check was found to be <u>O.K.</u>

E18-1.11.1.3 Minimum Reinforcement Check

Following the procedure in E18-1.7.1.4, the minimum reinforcement check was found to be <u>O.K.</u>

E18-1.11.2 Positive Moment Reinforcement for Span 2

<u>Examine the 0.5 point of span 2</u>

E18-1.11.2.1 Design for Strength

Following the procedure in E18-1.7.1.1, using Strength I Limit State:

$$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM}) \leq 0.90\, A_s\, f_s\, (d_s - a/2)$$

The positive live load moment shall be the largest caused by live loads (LL#1 or LL#2). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM).

From Table E18.5, the largest live load moment is from (LL#1), therefore at (0.5 pt.) of span 2:

M_{DC} = 24.0 kip-ft M_{DW} = 1.6 kip-ft M_{LL+IM} = 7.4 + 42.2 = 49.6 kip-ft

$M_u := 1.25\cdot(24.0) + 1.50\cdot(1.6) + 1.75\cdot(49.6)$ $\boxed{M_u = 119.2}$ kip-ft

$b := 12$ inches (for a one foot design width) and $\boxed{d_s = 14.9}$ in

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_s, are:

$\boxed{R_u = 597}$ psi $\qquad \boxed{\rho = 0.011} \qquad \boxed{A_s = 1.97}\ \frac{in^2}{ft}$

For Span 2:

A_s (req'd) = 1.97 $\frac{in^2}{ft}$ (to satisfy Exterior Strip requirements)

A_s (prov'd) = 2.00 $\frac{in^2}{ft}$ (#9 at 6" c-c spacing) (to satisfy Interior Strip requirements)

Therefore, <u>use: #9 at 6" c-c spacing</u> (A_s = 2.00 $\frac{in^2}{ft}$) in both Interior and Exterior Strips.

E18-1.11.2.2 Check Crack Control

Following the procedure in E18-1.7.1.3, the crack control check was found to be <u>O.K.</u>

E18-1.11.2.3 Minimum Reinforcement Check

Following the procedure in E18-1.7.1.4, the minimum reinforcement check was found to be <u>O.K.</u>

E18-1.11.3 Negative Moment Reinforcement at Piers

<u>Examine at C/L of Pier</u>

E18-1.11.3.1 Design for Strength

Following the procedure in E18-1.7.1.1, using Strength I Limit State:

$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM}) \leq 0.90\, A_s\, f_s\, (d_s - a/2)$

The negative live load moment shall be the largest caused by live loads (LL#1, LL#2 or LL#3). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM).

From Table E18.5, the largest live load moment is from (LL#2) and therefore at (C/L of Pier) :

M_{DC} = -72.6 kip-ft $\qquad M_{DW}$ = -4.9 kip-ft $\qquad M_{LL+IM}$ = -14.0 + (-45.0) = -59.0 kip-ft

$M_u := 1.25\cdot(-72.6) + 1.50\cdot(-4.9) + 1.75\cdot(-59.0) \qquad \boxed{M_u = -201.3}$ kip-ft

$b := 12$ inches (for a one foot design width) and $\boxed{d_s = 25.5}$ in

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_s, are:

$\boxed{R_u = 344}$ psi $\quad \boxed{\rho = 0.0061} \quad \boxed{A_s = 1.87} \ \frac{in^2}{ft}$

At C/L Pier:

$A_s \text{ (req'd)} = 1.87 \frac{in^2}{ft}$ (to satisfy Exterior Strip requirements)

$A_s \text{ (prov'd)} = 1.88 \frac{in^2}{ft}$ (#8 at 5" c-c spacing) (to satisfy Interior Strip requirements)

Therefore, <u>use: #8 at 5" c-c spacing</u> ($A_s = 1.88 \frac{in^2}{ft}$) in both Interior and Exterior Strips

E18-1.11.3.2 Check Crack Control

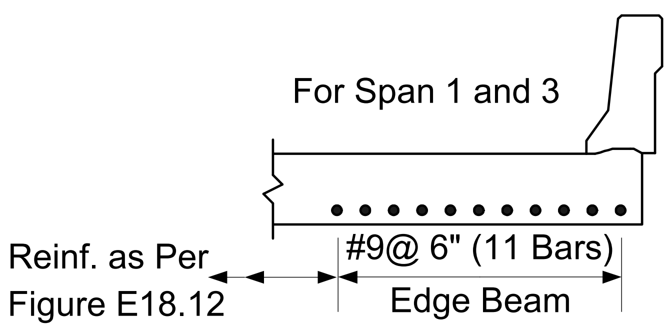
Following the procedure in E18-1.7.1.3, the crack control check was found to be <u>O.K.</u>

E18-1.11.3.3 Minimum Reinforcement Check

Following the procedure in E18-1.7.2.4, the minimum reinforcement check was found to be <u>O.K.</u>

<u>Edge Beam Reinforcement:</u>

The only location where Interior Strip reinforcement is not also placed in the Exterior Strip is in Span 1 and 3 for the bottom bars, as shown in Figure E18.13.



<u>Figure E18.13</u>

Exterior Strip Reinforcement

E18-1.11.4 Bar Steel Cutoffs

Select longitudinal reinforcement cutoff locations for an Exterior Strip.

Follow the procedure in E18-1.7.5, using reinforcement placed in the Exterior Strip. The cutoff locations must meet crack control requirements (fatigue criteria is not applied to an Exterior Strip).

E18-1.11.4.1 Span 1 Positive Moment Reinforcement (Cutoffs)

It was found that 1/2 of the bars in the Exterior Strip may be cut at 10.5 (ft) from the C/L of pier. The bars in the Interior Strip are to be cut at 10.0 (ft) from the C/L of pier. In order to use the same bar mark for this reinforcement, cut 1/2 of all positive reinforcement in the span at 10.0 (ft) from the C/L of pier, if crack control criteria in Exterior Strip is satisfied. The remaining bars follow the layout as shown in Figure E18.12.

E18-1.11.4.1.1 Check Crack Control

Following the procedure in E18-1.7.1.3, the crack control check was found to be O.K.

E18-1.11.4.2 Span 2 Positive Moment Reinforcement (Cutoffs)

It was found that 1/2 of the bars in the Exterior Strip may be cut at 11.0 (ft) from the C/L of pier. The bars in the Interior Strip are to be cut at 11.5 (ft) from the C/L of pier. In order to use the same bar mark for this reinforcement, cut 1/2 of all positive reinforcement in the span at 11.0 (ft) from the C/L of each pier, if crack control criteria in Exterior Strip is satisfied. The remaining bars follow the layout as shown in Figure E18.12.

E18-1.11.4.2.1 Check Crack Control

Following the procedure in E18-1.7.1.3, the crack control check was found to be O.K.

E18-1.11.4.3 Span 1 Negative Moment Reinforcement (Cutoffs)

It was found that 1/2 of the bars in the Exterior Strip may be cut at 15.5 (ft) from the C/L of pier. The bars in the Interior Strip are to be cut at 14.5 (ft) from the C/L of pier. In order to use the same bar mark for this reinforcement, cut 1/2 of all negative reinforcement in the span at 15.5 (ft) from the C/L of pier, if crack control criteria in Exterior Strip is satisfied. The remaining bars follow the layout as shown in Figure E18.12.

E18-1.11.4.3.1 Check Crack Control

Following the procedure in E18-1.7.1.3, the crack control check was found to be O.K.

E18-1.11.4.4 Span 2 Negative Moment Reinforcement (Cutoffs)

It was found that 1/2 of the bars in the Exterior Strip may be cut at 13.5 (ft) from the C/L of pier. The bars in the Interior Strip are to be cut at 13.0 (ft) from the C/L of pier. In order to use the same bar mark for this reinforcement, cut 1/2 of all negative reinforcement in the span at 13.5

(ft) from the C/L of pier, if crack control criteria in Exterior Strip is satisfied. The remaining bars follow the layout as shown in Figure E18.12.

E18-1.11.4.4.1 Check Crack Control

Following the procedure in E18-1.7.1.3, the crack control check was found to be O.K.

E18-1.12 Transverse Distribution Reinforcement

The criteria for main reinforcement parallel to traffic is applied. The amount of transverse distribution reinforcement (located in bottom of slab) is to be determined as a percentage of the main reinforcing steel required for positive moment **LRFD [5.12.2.1]**.

Spans 1 & 3:

$$\text{Percentage} = \frac{100\%}{\sqrt{L}} \le 50\% \ \text{Max.} \quad \text{(L is the span length in feet)}$$

Main positive reinforcement equals #9 at 7" c-c spacing $\left(A_s = 1.7\cdot\frac{in^2}{ft}\right)$

$$\text{Percentage} = \frac{100\%}{\sqrt{38}} = 16.2\% < 50\% \ \text{Max.}$$

$$A_s := 0.162\cdot(1.71) \qquad \boxed{A_s = 0.28}\ \frac{in^2}{ft}$$

Therefore, use #5 at 12" c-c spacing $A_s = 0.31\ \frac{in^2}{ft}$

Span 2:

Main positive reinforcement equals #9 at 6" c-c spacing $\left(A_s = 2.0(\frac{in^2}{ft}\right)$

$$\text{Percentage} = \frac{100\%}{\sqrt{51}} = 14.0\% < 50\% \ \text{Max.}$$

$$A_s := 0.140\cdot(2.00) \qquad \boxed{A_s = 0.28}\ \frac{in^2}{ft}$$

Therefore, use #5 at 12" c-c spacing $A_s = 0.31\ \frac{in^2}{ft}$

Refer to Standard 18.01 for placement of distribution reinforcement. For simplicity, the distribution reinforcement has been placed as shown in Figure E18.12.

E18-1.13 Shrinkage and Temperature Reinforcement Check

Check shrinkage and temperature reinforcement criteria for the reinforcement selected in preceeding sections.

E18-1.13.1 Longitudinal and Transverse Distribution Reinforcement

The area of reinforcement (A_s) per foot, for shrinkage and temperature effects, on each face and in each direction shall satisfy: **LRFD [5.10.6]**

$$A_s \geq \frac{1.30 \cdot b \cdot (h)}{2 \cdot (b + h) \cdot f_y} \qquad \text{and} \qquad 0.11 \leq A_s \leq 0.60$$

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

For cross-section of slab away from the haunch, the slab depth is 17 in., therefore:

$b := slab_{width}$ $\boxed{b = 510}$ in

$h := d_{slab}$ $\boxed{h = 17}$ in

$\boxed{f_y = 60}$ ksi

For each face, req'd A_s is:

$$A_s \geq \frac{1.30 \cdot (510) \cdot 17}{2 \cdot (510 + 17) \cdot 60} = 0.178 \ \frac{in^2}{ft}$$

, therefore, $0.11 \leq A_s \leq 0.60$ <u>O.K.</u>

For cross-section of slab at C/L of pier, the slab depth is 28 in., therefore:

$b := slab_{width}$ $\boxed{b = 510}$ in

$h := D_{haunch}$ $\boxed{h = 28}$ in

$\boxed{f_y = 60}$ ksi

For each face, req'd A_s is:

$$A_s \geq \frac{1.30 \cdot (510) \cdot 28}{2 \cdot (510 + 28) \cdot 60} = 0.288 \ \frac{in^2}{ft}$$

, therefore, $0.11 \leq A_s \leq 0.60$ <u>O.K.</u>

Shrinkage and temperature reinforcement shall not be spaced farther apart than 3.0 times the component thickness or 18 inches.

Max. spa = 3.0(17) = 51 in. or <u>18 in. governs</u>

In **LRFD [5.10.3.2]**, the maximum center to center spacing of adjacent bars is also 18 inches.

All longitudinal reinforcement (top/bottom) and transverse distribution reinforcement (bottom) in the slab exceeds A_s req'd. for each face, and does not exceed maximum spacing. O.K.

E18-1.14 Shear Check of Slab

Slab bridges designed for dead load and (HL-93) live load moments in conformance with **LRFD [4.6.2.3]** may be considered satisfactory in shear. O.K. per **LRFD [5.12.2.1]**

E18-1.15 Longitudinal Reinforcement Tension Check

Check the longitudinal reinforcement (in bottom of slab) located at the abutments for resistance to tension caused by shear **LRFD [5.7.3.5]**, using Strength I Limit State. Calculate shear from dead load and (HL-93) live load on interior and exterior strips. Assume a diagonal crack would start at the inside edge of the bearing area.

The concrete slab rests on an A1 (fixed) abutment, which has a width of 2.5 ft. For a 6 degree skew, the distance along the C/L of the bridge is 2.52 ft. Determine the distance D_{crack} from the end of the slab to the point at which the diagonal crack will intersect the bottom longitudinal reinforcement.

Assume the crack angle is: $\theta := 35$ degrees

The distance from the bottom of slab to the center of tensile reinforcement is 2.06 inches.

$$D_{crack} := (2.52) + \left(\frac{2.06}{12}\right)\cdot\frac{\cot(\theta)}{\cos(6)} \qquad \boxed{D_{crack} = 2.78} \text{ ft}$$

For an interior strip:

The longitudinal reinforcement provided is #9 at 7" c-c spacing ($1.71\ \frac{in^2}{ft}$)

The development length (ℓ_d) from (Table 9.9-2, Chapter 9) is 3'-9" (3.75 ft.)

The nominal tensile resistance (T_{nom}), of the longitudinal bars at the crack location is:

$$T_{nom} = A_s\cdot(f_y)\cdot\left[\frac{D_{crack} - (end\_cover)}{dev\_length}\right] \le A_s\cdot(f_y) = 102.6 \text{ kips}$$

$$T_{nom} := (1.71)\cdot 60.0\cdot\left(\frac{2.78\cdot 12 - 2}{3.75\cdot 12}\right) \qquad \boxed{T_{nom} = 71.5} \text{ kips}$$

The factored tension force (T_{fact}), from shear, to be resisted is from **LRFD [Eq'n. 5.7.3.5-2]**, where $V_s = V_p = 0$, is:

$$T_{fact} = \left(\frac{V_u}{\phi_v}\right)\cdot\cot(\theta)$$

Looking at E18-1.2: $\eta_i := 1.0$

and from Table E18.1: $\gamma_{DCmax} := 1.25$ $\gamma_{DWmax} := 1.50$ $\gamma_{LLstr1} := 1.75$ $\phi_v := 0.9$

$Q_i = V_{DC}, V_{DW}, V_{LL+IM}$ **LRFD [3.6.1.2, 3.6.1.3.3]**; shear due to applied loads as stated in E18-1.2

$Q = V_u = \eta_i\,[\gamma_{DCmax}(V_{DC}) + \gamma_{DWmax}(V_{DW}) + \gamma_{LLstr1}(V_{LL+IM})]$

$= 1.0\,[1.25(V_{DC}) + 1.50(V_{DW}) + 1.75(V_{LL+IM})]$

Therefore:

$V_u = 1.25(V_{DC}) + 1.50(V_{DW}) + 1.75(V_{LL+IM})$ (Factored Load Equation)

The live load shear shall be the largest caused by live loads (LL#1 or LL#2). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM).

From the computer analysis, for a one foot design width:

$V_{DC} = 2.96$ kip $V_{DW} = 0.3$ kip $V_{LL+IM} = 0.94 + 5.68 = 6.62$ kip (LL#2)

$V_u := 1.25\cdot(2.96) + 1.50\cdot(0.3) + 1.75\cdot(6.62)$ $\boxed{V_u = 15.74}$ kips (at C/L abutment)

$T_{fact} := \left(\frac{V_u}{\phi_v}\right)\cdot\cot(\theta)$ $\boxed{T_{fact} = 24.97}$ kips

Therefore: $T_{fact} = 24.97$ kips < $T_{nom} = 71.5$ kips O.K.

For simplicity, the value of V_u at the abutment centerline was used.

If the values for T_{fact} and T_{nom} were close, the procedure for determining the crack angle (θ) as outlined in **LRFD [5.7.3.4.2]** should be used.

The Exterior Strip was also examined and the longitudinal reinforcement was found to be satisfactory. O.K.

E18-1.16 Transverse Reinforcement in Slab over the Piers

The bridge in this example has a pier with (4) circular columns and a (2.5 ft x 2.5 ft) pier cap with rounded cap ends. (See Figure E18.14)

Out to out width of slab = $slab_{width}$ $\boxed{slab_{width} = 42.5}$ ft

Width of slab along skew = $slab_{skew} = \dfrac{42.5}{\cos(6deg)}$ $\boxed{slab_{skew} = 42.73}$ ft

Using a 6 inch offset from edge of slab to edge of pier cap per Standard 18.02 gives:

Length of Pier cap = cap_{length}

$$cap_{length} = 42.73 - 2\cdot\left(\frac{1.25 + 0.5}{\cos(6deg)} - 1.25\right) \qquad \boxed{cap_{length} = 41.71} \text{ ft}$$

E18-1.16.1 Dead Load Moments

Find the reaction, S_{DL} , (on a one foot slab width) at the pier due to (DC_{slab}) and ($DC_{1/2"WS}$). This dead load will be carried by the pier cap.

From the computer analysis, $S_{DL} := 12.4 \frac{kip}{ft}$ at the pier.

For a 2.5 ft by 2.5 ft pier cap: $Cap_{DL} := 1.0 \frac{kip}{ft}$

Therefore, the uniform dead load on the pier cap = PDL

$$PDL := S_{DL} + Cap_{DL} \qquad \boxed{PDL = 13.4} \frac{kip}{ft}$$

Calculate the dead load moments at columns (A,B,C & D), as shown in Figure E18.14, using the three-moment equation. The moments at columns (A & D) are equal, therefore:

$$M_A = \frac{1}{2}(PDL)\cdot L^2 \qquad M_A := \frac{1}{2}\cdot(13.4)\cdot 1.25^2 \qquad \boxed{M_A = 10.5} \text{ kip-ft} \qquad \boxed{M_D = 10.5} \text{ kip-ft}$$

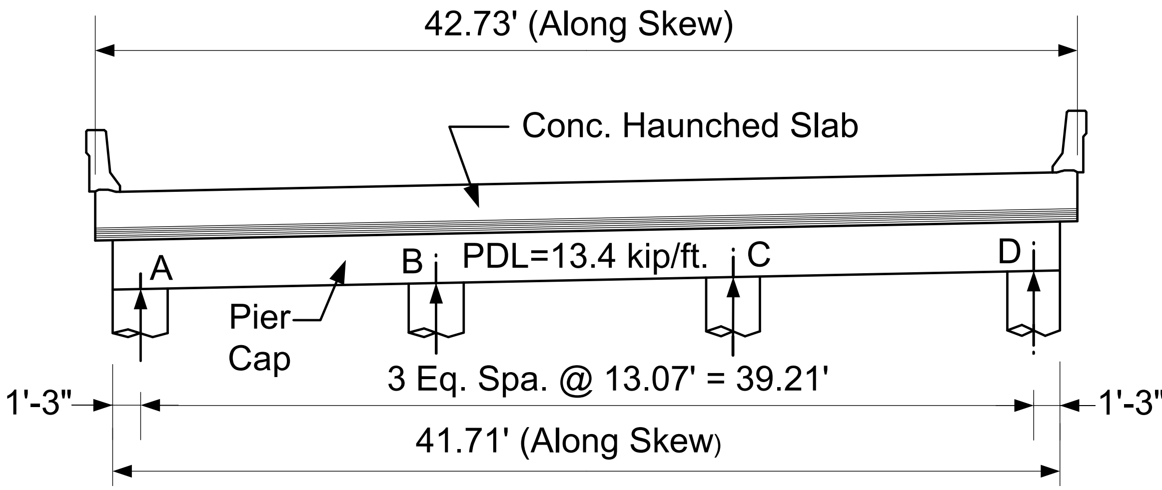


Figure E18.14

Section along C/L of Pier

Applying the three-moment equation for M_B gives values of:

$$\frac{6\cdot A\cdot a}{L} = \frac{6\cdot A\cdot b}{L} = \frac{(PDL)\cdot L^3}{4} \qquad \frac{(PDL)\cdot L^3}{4} = \frac{(13.4)\cdot 13.07^3}{4} = 7480 \ \text{kip-ft}$$

The three-moment equation is: $M_A\cdot L_1 + 2\cdot M_B\cdot\left(L_1 + L_2\right) + M_c\cdot L_2 + 6\cdot\frac{A_1\cdot a_1}{L_1} + 6\frac{A_2\cdot b_2}{L_2} = 0$

Refer to "Strength of Materials" textbook for derivation of the three-moment equation.

Other methods such as influence tables or moment distribution can also be used to obtain the dead load moments.

If M_A is known and due to symmetry $M_B = M_C$; the above equation reduces to one unknown, M_B, as follows:

$$(-10.5)\cdot 13.07 + 2\cdot M_B\cdot(13.07 + 13.07) + M_B\cdot(13.07) + 7480 + 7480 = 0$$

Therefore, solving for M_B and knowing $M_C = M_B$: $\boxed{M_B = 226.8}$ kip-ft $\boxed{M_C = 226.8}$ kip-ft

Find the reaction (on a one foot slab width) at the pier due to (DC_{FWS}) and (DC_{para}). This dead load will be carried by the pier cap and a transverse beam represented by a portion of the slab over the pier.

From the computer analysis, FWS + para. (DL) = 1.9 $\frac{kip}{ft}$ at the pier.

Using the three-moment equation, $\boxed{M_A = 1.5}$ kip-ft $\boxed{M_D = 1.5}$ kip-ft

$\boxed{M_B = 32.2}$ kip-ft $\boxed{M_C = 32.2}$ kip-ft

The partial dead load moment diagram for "PDL" and "FWS + para (DL)" is shown in Figure E18.15.

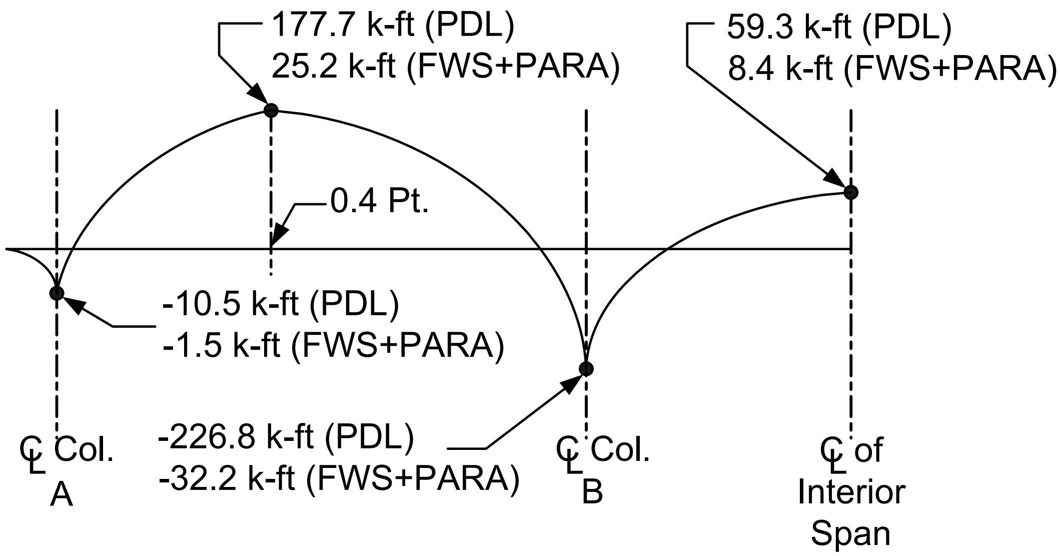


Figure E18.15

Dead Load Moment Diagram

E18-1.16.2 Live Load Moments

The maximum live load reactions at the pier shall be the largest caused by live loads (LL#1, LL#2 or LL#3). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM).

The reactions per lane (from computer analysis), before applying the dynamic load allowance (IM) are:

Design Lane Load = <u>35.1 kips</u>

Design Tandem = 50.0 kips

Design Truck = <u>68.9 kips</u>

90% Double Design Trucks = 62.1 kips

90% Design Lane Load = 31.6 kips

The largest live load reaction is from: Design Truck + Design Lane Load (LL#2)

The dynamic load allowance (IM) is 33% .

Design Truck Reaction (including IM = 33%):

$$1.33 \cdot (68.9) = 91.64 \ \frac{\text{kip}}{\text{truck}} \text{, therefore, Wheel Load} = \frac{91.64}{2} = 45.8 \ \frac{\text{kip}}{\text{wheel}}$$

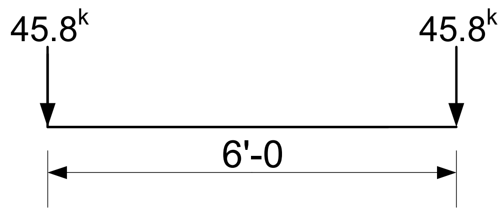


Figure E18.16

Design Truck Reaction

Design Lane Load Reaction (IM not applied to Lane Load):

$$\frac{(35.1)\,kip}{(10)\_ft\,lane} = 3.51\,\frac{kip}{ft}$$

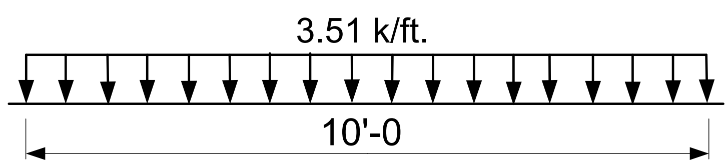


Figure E18.17

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.

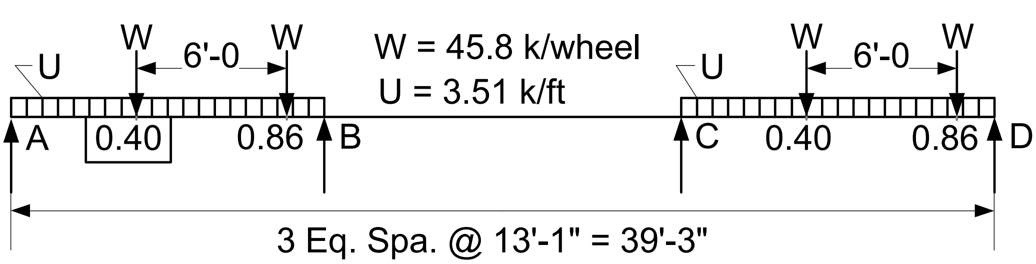


Figure E18.18

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, 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.

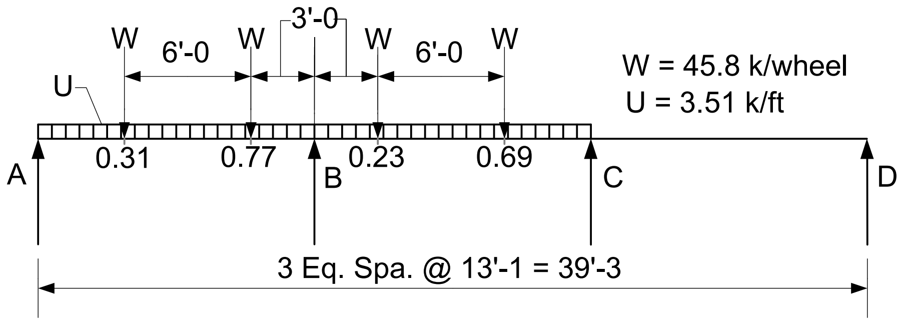


Figure E18.19

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 [5.7.4]** 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):

b_{neg} = width of pier cap = 2.5 ft $\boxed{b_{neg} = 30}$ in

d_{neg} = D_{haunch} + cap depth - top clr. - top bar dia. - 1/2 bar dia.

$d_{neg} := 28 + 30 - 2 - 1 - 0.38$ $\boxed{d_{neg} = 54.62}$ in

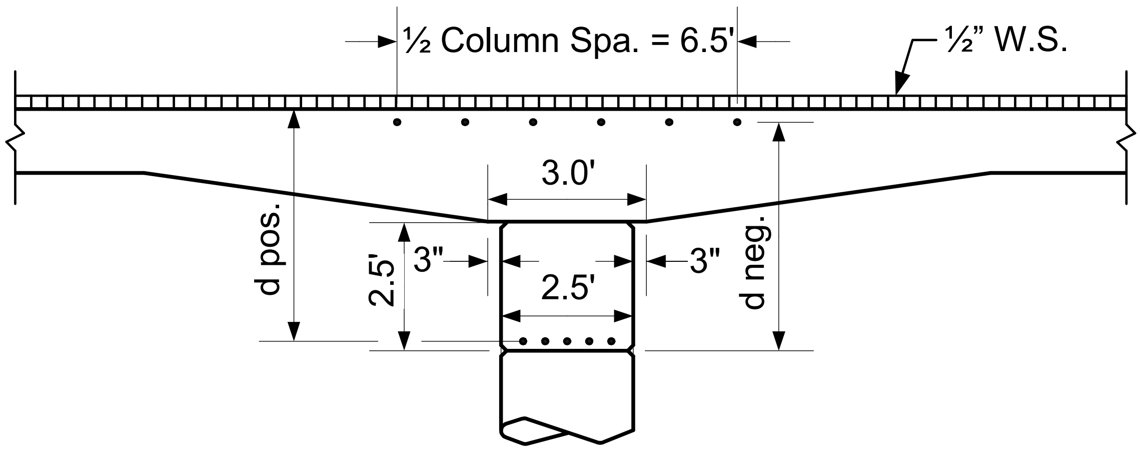


Figure E18.20

Details for Transverse Slab Member

E18-1.16.3 Positive Moment Reinforcement for Pier Cap

Examine the 0.4 point of the Exterior span

E18-1.16.3.1 Design for Strength

The dead load, PDL, carried by the pier cap is from (DC_{slab}) + ($DC_{1/2"WS}$) + Pier Cap DL.

Following the procedure in E18-1.7.1.1, using Strength I Limit State:

$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM})$

M_{DC} = 177.7 kip-ft (See Figure E18.15)

$M_u := 1.25 \cdot (177.7)$ (contribution from PDL) $\boxed{M_u = 222.1}$ kip-ft

b_{cap} **=** 2.5ft (pier cap width) $\boxed{b_{cap} = 30}$ in

d_s = pier cap depth - bott. clr. - stirrup dia. - 1/2 bar dia.

$d_s := 30 - 1.5 - 0.625 - 0.44$ $\boxed{d_s = 27.43}$ in

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_{s1}, are:

$\boxed{R_u = 131}$ psi $\boxed{\rho = 0.00223}$ $\boxed{A_{s1} = 1.84}$ in^2

The dead loads (FWS + para DL) and live load (LL+IM) are carried by the pier cap and the transverse slab member acting as a unit.

Split the (FWS + para DL) dead load moment (from Figure E18.15) into components:

$M_{DC} := 11.9$ kip-ft (moment from para DL)

$M_{DW} := 13.3$ kip-ft (moment from FWS)

$M_{LL+IM} = 210.1$ kip-ft

$M_u := 1.25\cdot(11.9) + 1.50\cdot(13.3) + 1.75\cdot(210.1)$ $\boxed{M_u = 402.5}$ kip-ft

$\boxed{b_{pos} = 78}$ in (See E18-1.16.2)

$\boxed{d_{pos} = 55.44}$ in (See E18-1.16.2)

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_{s2}, are:

$\boxed{R_u = 22.4}$ psi $\boxed{\rho = 0.00037}$ $\boxed{A_{s2} = 1.6}$ in^2

$A_{s\_total} := A_{s1} + A_{s2}$ $\boxed{A_{s\_total} = 3.44}$ in^2

E18-1.16.4 Negative Moment Reinforcement for Pier Cap

Examine at C/L of Column "B"

E18-1.16.4.1 Design for Strength

The dead load, PDL, carried by the pier cap is from (DC_{slab}) + ($DC_{1/2\text{"}WS}$) + Pier Cap DL.

Following the procedure in E18-1.7.1.1, using Strength I Limit State:

$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM})$

$M_{DC} = 226.8$ kip-ft (See Figure E18.15)

$M_u := 1.25\cdot(226.8)$ (contribution from PDL) $\boxed{M_u = 283.5}$ kip-ft

$\boxed{b_{cap} = 30}$ in (pier cap width)

d_s = pier cap depth - top clr. - stirrup dia. - 1/2 bar dia.

$d_s := 30 - 1.5 - 0.625 - 0.44$ $\boxed{d_s = 27.43}$ in

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_s, are:

$\boxed{R_u = 167}$ psi $\boxed{\rho = 0.00286}$ $\boxed{A_s = 2.35}$ in^2

E18-1.16.5 Positive Moment Reinforcement for Transverse Slab Member

See Standard 18.01 for minimum reinforcement at this location

E18-1.16.6 Negative Moment Reinforcement for Transverse Slab Member

Examine at C/L of Column "B"

E18-1.16.6.1 Design for Strength

The dead loads (FWS + para DL) and live load (LL+IM) are carried by the pier cap and the transverse slab member acting as a unit.

Following the procedure in E18-1.7.1.1, using Strength I Limit State:

$$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM})$$

Split the (FWS + para DL) dead load moment (from Figure E18.15) into components:

$M_{DC} := 15.2$ kip-ft (moment from para DL)

$M_{DW} := 17.0$ kip-ft (moment from FWS)

$M_{LL+IM} = 234.7$ kip-ft

$M_u := 1.25\cdot(15.2) + 1.50\cdot(17.0) + 1.75\cdot(234.7)$ $\boxed{M_u = 455.2}$ kip-ft

$\boxed{b_{neg} = 30}$ in (See E18-1.16.2)

$\boxed{d_{neg} = 54.62}$ in (See E18-1.16.2)

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_s, are:

$\boxed{R_u = 67.8}$ psi $\boxed{\rho = 0.00114}$ $\boxed{A_s = 1.87}$ in^2

In E18-1.16.8, check to see if this bar area meets the minimum reinforcement criteria. Then the bar size and spacing can be selected.

E18-1.16.7 Shear Check of Slab at the Pier

Check the shear (reaction) in the slab at the pier, using Strength I Limit State.

Due to the geometry and loading, stirrups are generally not required or recommended.

Looking at E18-1.2: $\eta_i := 1.0$

and from Table E18.1: $\gamma_{DCmax} := 1.25$ $\gamma_{DWmax} := 1.50$ $\gamma_{LLstr1} := 1.75$ $\phi_v := 0.9$

$Q_i = V_{DC}, V_{DW}, V_{LL+IM}$ **LRFD [3.6.1.2, 3.6.1.3.3]**; shear (reactions) due to applied loads as stated in E18-1.2

$Q = V_u = \eta_i\,[\gamma_{DCmax}(V_{DC}) + \gamma_{DWmax}(V_{DW}) + \gamma_{LLstr1}(V_{LL+IM})]$

$= 1.0\,[1.25(V_{DC}) + 1.50(V_{DW}) + 1.75(V_{LL+IM})]$

$V_r = \phi_v \cdot V_n$

Therefore: $V_u \le V_r$ (Limit States Equation)

$V_u = 1.25(V_{DC}) + 1.50(V_{DW}) + 1.75(V_{LL+IM}) \le \phi_v\, V_n = V_r$

Find the dead load reactions at the Pier:

From the computer analysis, for a one foot design width:

V_{DC1} = reaction from $(DC_{slab}) + (DC_{1/2"WS})$ = 12.4 kip/ft

V_{DC2} = reaction from (DC_{para}) = 0.9 kip/ft

Therefore, total reaction (V_{DC}) from these loads across the slab width is:

$V_{DC} := (12.4 + 0.9)\cdot 42.5$ $\boxed{V_{DC} = 565.3}$ kips

V_{DW} = reaction from (DW_{FWS}) future wearing surface = 1.0 kip/ft

Therefore, total reaction (V_{DW}) from this load across the slab width is:

$V_{DW} := 1.0\cdot(42.5)$ $\boxed{V_{DW} = 42.5}$ kips

Find the live load reaction at the Pier:

For live load, use (3) design lanes **LRFD [3.6.1.1.1]** and multiple presence factor (m = 0.85) **LRFD [3.6.1.1.2]**.

From E18-1.16.2: Design Truck Reaction= $91.64\ \dfrac{kip}{truck}$ (for one lane)

Design Lane Load Reaction= $35.1\ \dfrac{kip}{lane}$ (for one lane)

Therefore, total reaction (V_{LL+IM}) from these loads is:

$V_{LL+IM} = (91.64 + 35.1)(\text{ 3 design lanes})(0.85)$ $\boxed{V_{LL+IM} = 323.2}$ kips

$V_u := 1.25\cdot(565.3) + 1.50\cdot(42.5) + 1.75\cdot(323.2)$ $\boxed{V_u = 1336}$ kips

Check for shear (two-way action): **LRFD [5.12.8.6.3]**

$$V_r = \phi_v \cdot V_n = \phi_v \cdot \left(0.063 + \frac{0.126}{\beta_c}\right) \cdot \lambda \sqrt{f'_c} \cdot (b_o) \cdot (d_v) \le \phi_v \cdot (0.126) \cdot \lambda \sqrt{f'_c} \cdot (b_o) \cdot (d_v)$$

Where:

β_c = ratio of long side to short side of the rectangle through which reaction force is transmitted $\approx$ 41.71 ft. / 2.5 ft. = 16.7

d_v = effective shear depth = dist. between resultant tensile & compressive forces $\approx$ 24 in.

b_o = perimeter of the critical section $\approx$ 1109 in.

λ = concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

Therefore, $V_r := \phi_v \cdot \left(0.063 + \frac{0.126}{\beta_c}\right) \cdot \sqrt{f'_c} \cdot (b_o) \cdot d_v$ $\boxed{V_r = 3380}$ kips

but $\le \phi_v \cdot 0.126 \cdot \sqrt{f'_c} \cdot (b_o) \cdot d_v = 6036$ kips

Therefore, V_u = 1336 kips < V_r = 3380 kips O.K.

Note: Shear check and shear reinforcement design for the pier cap is not shown in this example. Also crack control criteria, minimum reinforcement checks, and shrinkage and temperature reinforcement checks are not shown for the pier cap.

E18-1.16.8 Minimum Reinforcement Check for Transverse Slab Member

Check the negative moment reinforcement (at interior column) for minimum reinforcement criteria.

The amount of tensile reinforcement shall be adequate to develop a factored flexural resistance (M_r), or moment capacity, at least equal to the lesser of: **LRFD [5.6.3.3]**

M_{cr} (or) $1.33M_u$

from E18-1.7.1.4, $M_{cr} = 1.1(f_r)\frac{I_g}{c}$

Where:

$f_r = 0.24\lambda\sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r = 0.24\sqrt{4}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.48}$ ksi

h = pier cap depth + D_{haunch} (section depth) $\boxed{h = 58}$ in

b_{cap} = pier cap width $\boxed{b_{cap} = 30}$ in

$I_g := \frac{1}{12} \cdot b_{cap} \cdot h^3$ (gross moment of inertia) $\boxed{I_g = 487780}$ in^4

$c := \frac{h}{2}$ (section depth/2) $\boxed{c = 29}$ in

$$M_{cr} = \frac{1.1 f_r \cdot (I_g)}{c} = \frac{1.1 \cdot 0.48 \cdot (487780)}{29(12)}$$ $\boxed{M_{cr} = 740.1}$ kip-ft

$1.33 \cdot M_u = 605.4$ kip-ft , where M_u was calculated for Strength Design in E18-1.16.6.1 and (M_u = 455.2 kip-ft)

1.33 M_u controls because it is less than M_{cr}

Recalculating requirements for (New moment = $1.33 \cdot M_u$ = 605.4 kip-ft)

$\boxed{b_{neg} = 30}$ in (See E18-1.16.2)

$\boxed{d_{neg} = 54.62}$ in (See E18-1.16.2)

Calculate R_u, coefficient of resistance:

$$R_u = \frac{M_u}{\phi_f \cdot (b_{neg}) \cdot d_{neg}^2} \qquad R_u := \frac{605.4 \cdot (12) \cdot 1000}{0.9(30) \cdot 54.62^2}$$ $\boxed{R_u = 90.2}$ psi

Solve for ρ, reinforcement ratio, using Table 18.4-3 (R_u vs ρ) in 18.4.13;

$\rho := 0.00152$

$A_s = \rho \cdot (b_{neg}) \cdot d_{neg}$ $\quad A_s := 0.00152 \cdot (30) 54.62$ $\boxed{A_s = 2.49}$ in^2

Place this reinforcement in a width, centered over the pier, equal to 1/2 the center to center column spacing or 8 feet, whichever is smaller. Therefore, width equals 6.5 feet.

Therefore, 2.49 in^2/6.5 ft. = 0.38 in^2/ft. Try <u>#5 at 9" c-c spacing</u> for a 6.5 ft. transverse width over the pier. This will provide (A_s = 2.79 in^2) in a 6.5 ft. width.

Calculate the depth of the compressive stress block

Assume $f_s = f_y$ (See 18.3.3.2.1) ; for f'_c = 4.0 ksi : $\alpha_1 := 0.85$ and $\beta_1 = 0.85$

$$a = \frac{A_s \cdot f_y}{\alpha_1 \cdot f'_c \cdot b_{neg}} \qquad a := \frac{2.79 \cdot (60)}{0.85 \cdot (4.0) \cdot 30}$$ $\boxed{a = 1.64}$ in

If $\frac{c}{d_s} \le 0.6$ for (f_y = 60 ksi) **LRFD [5.6.2.1]**, then reinforcement has yielded and the assumption is correct.

$\beta_1 := 0.85$ $\qquad c := \frac{a}{\beta_1}$ $\qquad \boxed{c = 1.93}$ in

$d_s := d_{neg}$ $\qquad \boxed{d_s = 54.62}$ in

$\frac{c}{d_s} = 0.04 < 0.6$ therefore, the reinforcement will yield.

$$M_r = 0.90 \cdot A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right)$$

$$M_r := 0.9 \cdot (2.79) \cdot 60.0 \cdot \left(\frac{54.62 - \frac{1.64}{2}}{12}\right) \qquad \boxed{M_r = 675.5} \text{ kip-ft}$$

Therefore, $1.33(M_u)$ = 605.4 kip-ft < M_r = 675.5 kip-ft O.K.

E18-1.16.9 Crack Control Check for Transverse Slab Member

Check the negative moment reinforcement (at interior column).

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**.

Following the procedure in E18-1.7.1.3, using Service I Limit State:

$\boxed{f_r = 0.48}$ ksi $\qquad \boxed{f_{r80\%} = 0.38}$ ksi $\qquad \boxed{c = 29}$ in $\qquad \boxed{I_g = 487780}$ in^4

$$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$$

Using same moments selected for Strength Design in E18-1.16.6, at (interior column), provides

M_{DC} = 15.2 kip-ft $\qquad M_{DW}$ = 17.0 kip-ft $\qquad M_{LL+IM}$ = 234.7 kip-ft

$$M_s := 1.0 \cdot (15.2) + 1.0(17.0) + 1.0 \cdot (234.7) \qquad \boxed{M_s = 266.9} \text{ kip-ft}$$

$$f_T = \frac{M_s \cdot c}{I_g} \qquad f_T := \frac{266.9 \cdot (29) \cdot 12}{487780} \qquad \boxed{f_T = 0.19} \text{ ksi}$$

f_T =0.19 ksi < 80% f_r = 0.38 ksi; therefore, crack control criteria check is not req'd.

Therefore, crack control criteria for transverse slab reinforcement is O.K.

Use: #5 at 9" c-c spacing for a 6.5 ft. transverse width over the pier.

The transverse slab member reinforcement (top/bottom), and the remainder of the transverse reinforcement is shown in Figure E18.21.

E18-1.17 Shrinkage and Temperature Reinforcement Check

Check shrinkage and temperature reinforcement criteria for the remaining transverse reinforcement.

E18-1.17.1 Transverse Slab Member and Other Transverse Reinforcement

Following the procedure in E18-1.13.1:

All transverse slab member reinforcement (top/bottom) and remainder of transverse reinforcement in slab exceeds A_s req'd. for each face, and does not exceed maximum spacing.

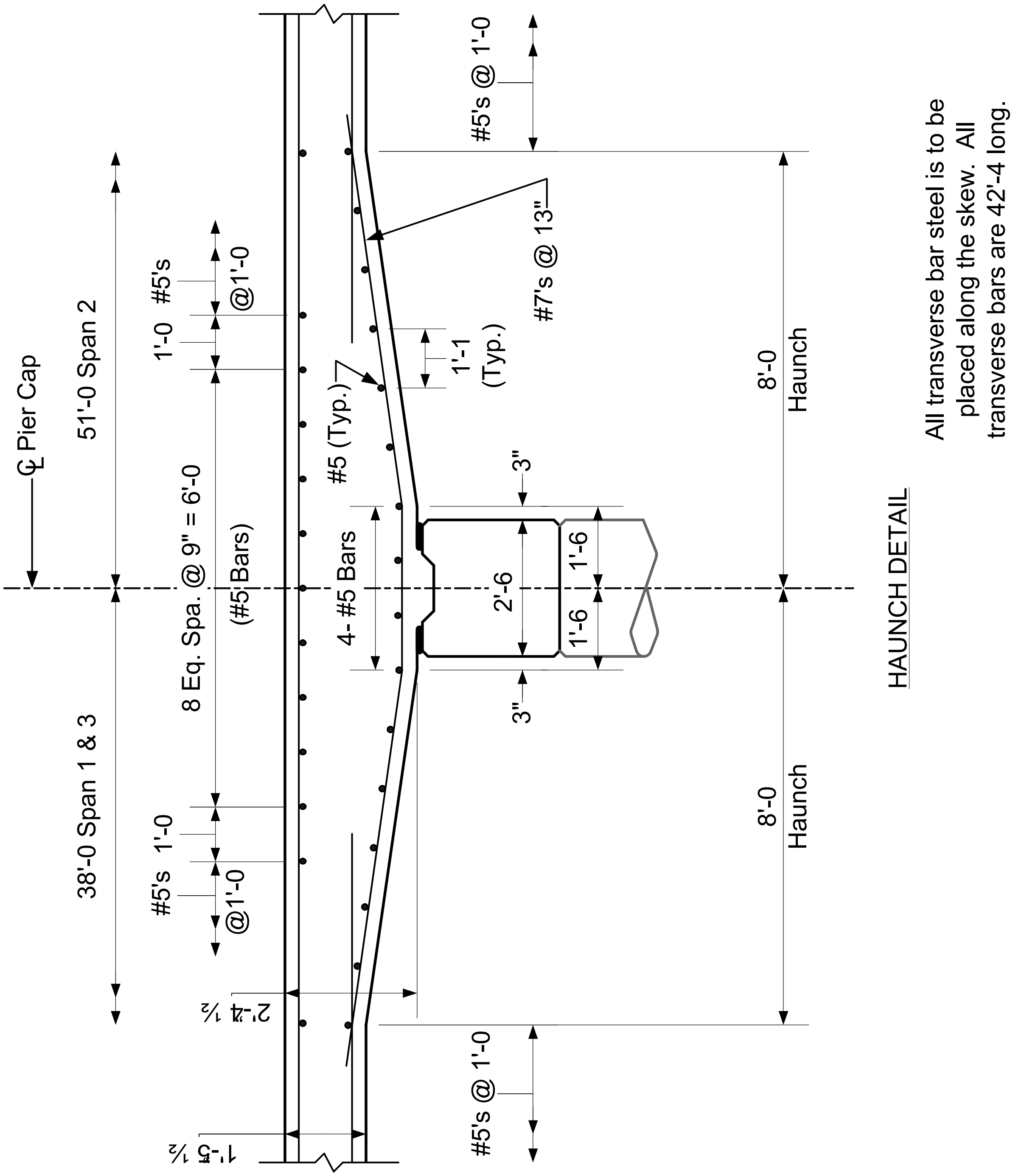


Figure E18.21

Haunch Detail

E18-1.18 Check for Uplift at Abutments

Check for uplift at the abutments, using Strength I Limit State **LRFD [C3.4.1, 5.5.4.3, 14.6.1]**

The maximum uplift at the abutments from live load is obtained from the following influence line and shall be the largest caused by live loads (LL#1 or LL#2) in each design lane (See Figure E18.22). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM).

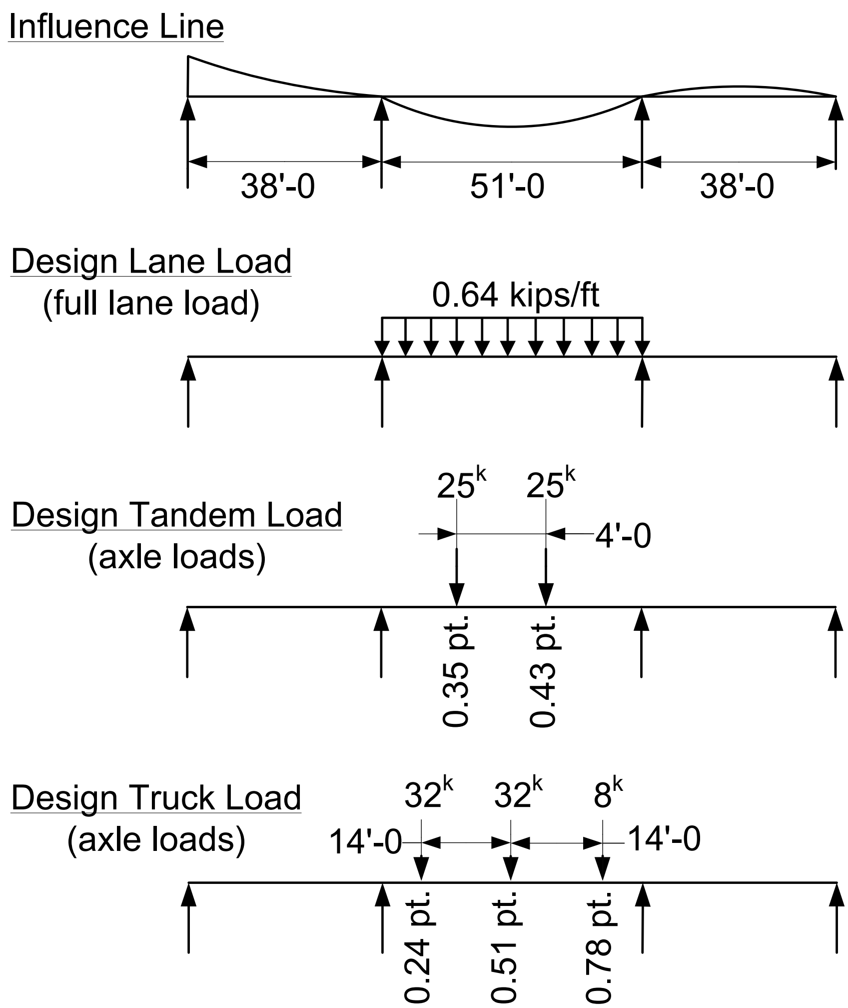


Figure E18.22

Influence Line and Live Loads for Uplift

Tables of influence line coefficients are used to calculate live load reactions at the abutment. The reactions per lane, before applying the dynamic load allowance (IM) are:

Design Lane Load Reaction = (0.1206)(0.64)(38.0) = <u>2.9 kips</u>

Design Truck Reaction = (0.1290 + 0.1360)(32) + (0.060)(8) = <u>9.0 kips</u>

Design Tandem Reaction = (0.149 + 0.148)(25) = 7.4 kips

The largest live load reaction is from: Design Truck + Design Lane Load (LL#2)

The dynamic load allowance (IM) is 33% ; (applied to Design Truck)

Therefore, total live load reaction (R_{LL+IM}) from these loads is:

R_{LL+IM} = 9.0(1.33) + 2.9 = 14.87 kips (for one lane)

Find the dead load reactions at the abutment:

From the computer analysis, for a one foot design width:

R_{DC1} = reaction from (DC_{slab}) + ($DC_{1/2"WS}$) = 2.8 kip/ft

R_{DC2} = reaction from (DC_{para}) = 0.3 kip/ft

Therefore, total dead load reaction (R_{DC}) from these loads across the slab width is:

$R_{DC} := (2.8 + 0.3)\cdot 42.5$ $\boxed{R_{DC} = 131.75}$ kips

Total dead load reaction ignores (DW_{FWS}) because it reduces uplift.

Check uplift for Strength I Limit State:

Looking at E18-1.2: $\eta_i := 1.0$ and Table E18.1: $\gamma_{DCmin} := 0.90$ $\gamma_{LLstr1} := 1.75$

Dead Load Reaction at Abutments = γ_{DCmin}(R_{DC}) = 0.90(131.75) = 118.6 kips

Uplift from Live Load = γ_{LLstr1}(R_{LL+IM})(# lanes loaded)(m)

Use (3) design lanes **LRFD [3.6.1.1.1]** and multiple presence factor (m = 0.85) **LRFD [3.6.1.1.2]**

Uplift from Live Load = 1.75(14.87)(3 design lanes)(0.85) = 66.4 kips

Therefore, Uplift = 66.4 kips < Dead Load Reaction = 118.6 kips O.K.

Because dead load reaction at abutments exceeds uplift from live load, the existing dowels (#5 at 1'-0 spa.) are adequate. (See Standard 12.01)

E18-1.19 Deflection Joints and Construction Joints

Locate deflection joints for concrete slab structures according to Standard 30.07. Refer to Standards 18.01/18.02 for recommended construction joint guidelines.

Note: See Standard 18.01/18.02 for required notes and other details

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Table of Contents

19.1 Introduction

This chapter provides information intended for prestressed I-girders. Prestressed box girders and general prestressed concrete guidelines are also included in this chapter.

The definition of prestressed concrete as given by the ACI Committee on Prestressed Concrete is:

> "Concrete in which there has been introduced internal stresses of such magnitude and distribution that the stresses resulting from given external loadings are counteracted to a desired degree. In reinforced concrete members the prestress is commonly introduced by tensioning the steel reinforcement."

This internal stress is induced into the member by either of the following prestressing methods.

19.1.1 Pretensioning

In pretensioning, the tendons are first stressed to a given level and then the concrete is cast around them. The tendons may be composed of wires, bars or strands.

The most common system of pretensioning is the long line system, by which a number of units are produced at once. First the tendons are stretched between anchorage blocks at opposite ends of the long stretching bed. Next the spacers or separators are placed at the desired member intervals, and then the concrete is placed within these intervals. When the concrete has attained a sufficient strength, the steel is released and its stress is transferred to the concrete via bond.

19.1.2 Post-Tensioning

In post-tensioning, the concrete member is first cast with one or more post-tensioning ducts or tubes for future insertion of tendons. Once the concrete is sufficiently strong, the tendons are stressed by jacking against the concrete. When the desired prestress level is reached, the tendons are locked under stress by means of end anchorages or clamps. Subsequently, the duct is filled with grout to protect the steel from corrosion and give the added safeguard of bond.

In contrast to pretensioning, which is usually incorporated in precasting (casting away from final position), post-tensioning lends itself to cast-in-place construction.

19.2 Basic Principles

This section defines the internal stress that results from either prestressing method.

First consider the simple beam shown in Figure 19.2-1.

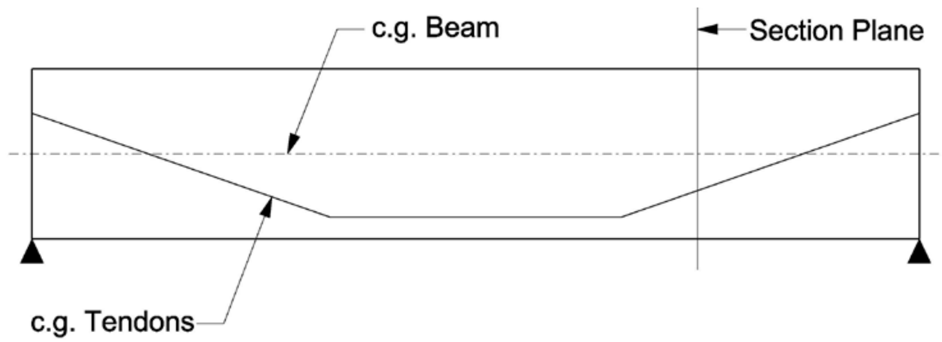

Figure 19.2-1
Simple Span Prestressed Concrete Beam

The horizontal component, P, of the tendon force, F, is assumed constant at any section along the length of the beam.

Also, at any section of the beam the forces in the beam and in the tendon are in equilibrium. Forces and moments may be equated at any section.

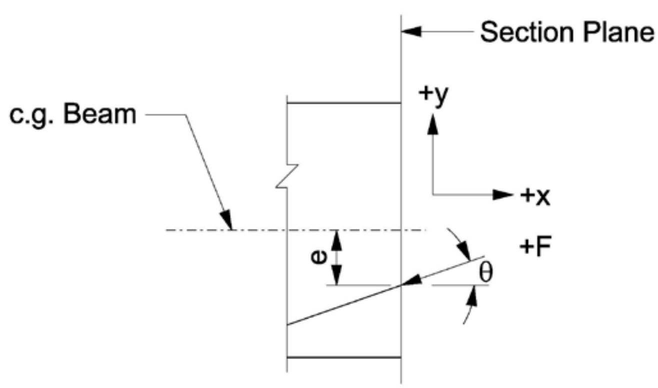

Figure 19.2-2
Assumed Sign Convention for Section Forces

The assumed sign convention is as shown in Figure 19.2-2 with the origin at the intersection of the section plane and the center of gravity (centroidal axis) of the beam. This convention indicates compression as positive and tension as negative.

The eccentricity of the tendon can be either positive or negative with respect to the center of gravity; therefore it is unsigned in the general equation. The reaction of the tendon on the beam is always negative; therefore the horizontal component is signed as:

$$P = F\cos\theta$$

Then, by equating forces in the x-direction, the reaction, P, of the tendon on the concrete produces a compressive stress equal to:

$$f_1 = \frac{P}{A}$$

Where:

A	=	Cross-sectional area of the beam

Since the line of action of the reaction, P, is eccentric to the centroidal axis of the beam by the amount e, it produces a bending moment.

$$M = Pe$$

This moment induces stresses in the beam given by the flexure formula:

$$f_2 = \frac{My}{I} = \frac{Pey}{I}$$

Where:

y	=	Distance from the centroidal axis to the fiber under consideration, with an unsigned value in the general equations
I	=	Moment of inertia of the section about its centroidal axis

The algebraic sum of f_1 and f_2 yields an expression for the total prestress on the section when the beam is not loaded.

$$f_p = f_1 + f_2 = \frac{P}{A} + \frac{Pey}{I}$$

Now, by substituting $I = Ar^2$, where r is the radius of gyration, into the above expression and arranging terms, we have:

$$f_p = \frac{P}{A}\left(1 + \frac{ey}{r^2}\right)$$

These stress conditions are shown in Figure 19.2-3.

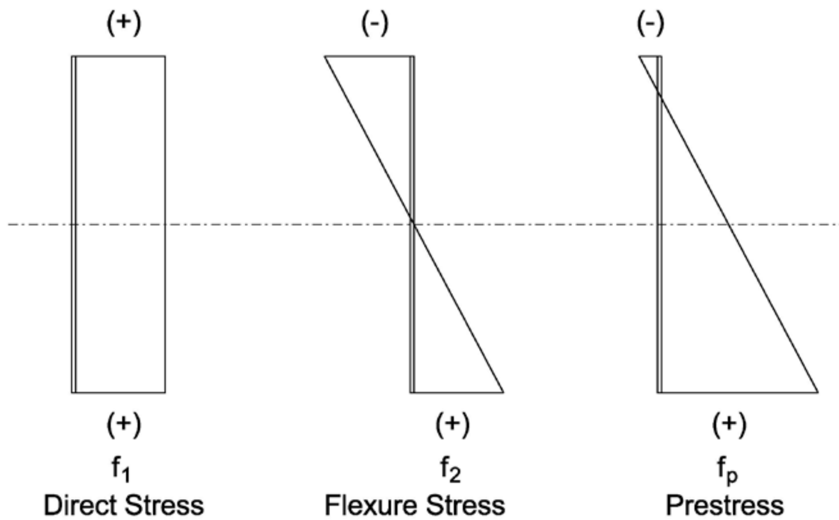

Figure 19.2-3
Calculation of Concrete Stress Due to Prestress Force

Finally, we equate forces in the y-direction which yields a shear force, V, over the section of the beam due to the component of the tendon reaction.

$$V = F\sin\theta = P\tan\theta$$

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 $f'_c \leq 6.8$ ksi
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:

For prestressed I-girders, 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.

For prestressed box girders, the use of concrete with strength greater than 5 ksi is only allowed with prior approval of the BOS Development Section.

The use of 8 ksi concrete for prestressed 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 prestressed 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 [5.9.2.2]** and **LRFD [5.9.2.3]**.

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 [5.9.3] 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.

WisDOT policy item:

WisDOT policy is to use the approximate method described in **LRFD [5.9.3.3]** 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 [5.9.3.2.3a]**, 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 [5.9.3.3],** 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 2.4 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 non-standard members with unusual dimensions or built using staged segmental construction, the refined method of **LRFD [5.9.3.4]** shall be used. For prestressed box girders time-dependent losses shall be determined using the refined method of **LRFD [5.9.3.4]**.

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 prestressed I-girder and prestressed box girder load-carrying behavior described in the next two sections.

19.3.2.3.1 Prestressed I-Girder

In the case of a prestressed 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, unshored 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 [5.12.3.3.5]** 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 Prestressed Box Girder

In the case of prestressed box girders with a thin concrete overlay, 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 composite section, the dead load of the deck is carried by the basic section and the live load, dynamic load allowance and any superimposed dead loads are carried by the composite section. A composite section shall consist of a reinforced deck, 6” minimum thickness, with composite shear reinforcement extending into the deck.

WisDOT policy item:

The use of prestressed box girders is subject to prior-approval by the Bureau of Structures. These structures are currently limited to the following requirements:

- Single spans
- Composite section details (design and rating based on non-composite section)
- 30 degree maximum skew
- AADT < 3,500 on non-NHS roadways

Variations to these requirements require approval by the Bureau of Structures.

19.3.2.4 Factored Flexural Resistance

At the final stage, the factored flexural resistance of the composite section is considered. Since the member is designed on a service load basis, it must be checked for its factored flexural resistance at the Strength I limit state. See section 17.2.3 for a discussion on limit states.

The need for both service load and strength computations lies with the radical change in a member's behavior when cracks form. Prior to cracking, the gross area of the member is effective. As a crack develops, all the tension in the concrete is picked up by the reinforcement. If the percentage of reinforcement is small, there is very little added capacity between cracking and failure.

19.3.2.5 Fatigue Limit State

At the final stage, the member is checked for the Fatigue I limit state. See section 17.2.3 for a discussion on limit states. Allowable compressive stresses in the concrete and tensile stresses in the non-prestressed reinforcement are checked.

19.3.3 Design Procedure

The intent of this section is to provide the designer with a general outline of steps for the design of pretensioned members. Sections of interest during design include, but are not limited to, the following locations:

- 10^{th} points
- Hold-down points

- Regions where the prestress force changes (consider the effects of transfer and development lengths, as well as the effects of debonded strands)

- Critical section(s) for shear

The designer must consider the amount of prestress force at each design section, taking into account the transfer length and development length, if appropriate.

19.3.3.1 Prestressed I-Girder Member Spacing

A trial prestressed I-girder arrangement is made by using Table 19.3-1 and Table 19.3-2 as a guide. An ideal spacing results in equal strands for interior and exterior girders, together with an optimum slab thickness. Current practice is to use a minimum haunch of (1-1/4" plus deck cross slope times one-half top flange width) for section property calculations and then use a 3" average haunch for concrete preliminary quantity calculations. After preliminary design this value should be revised as needed as outlined in 19.3.4. The maximum slab overhang dimensions are detailed in 17.6.2.

For prestressed I-girder bridges, other than pedestrian or other unusual structures, four or more girders shall be used.

19.3.3.2 Prestressed Box Girder Member Spacing

The prestressed box girder is used in an adjacent multi-beam system only. Precast units are placed side by side and locked (post-tensioned) together. The span length, desired roadway width and live loading control the size of the member.

When selecting a 3' wide section vs. 4' wide section, do not mix 3' wide and 4' wide sections across the width of the bridge. Examine the roadway width produced by using all 3' wide sections or all 4' wide sections and choose the system that is the closest to but greater than the required roadway width. While 3' wide sections may produce a slightly narrower roadway width 4' wide sections are still preferred since they require fewer sections. Verify the required roadway width is possible when considerations are made for the roadway cross-slope. Table 19.3-3 states the approximate span limitations for each section depth. Coordinate roadway width with roadway designers and consider some variability. See the Standards for prestressed box girder details.

19.3.3.3 Dead Load

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

The dead load moments and shears due to the girder and concrete deck are computed for simple spans. When superimposed dead loads are considered, the superimposed dead load moments are based on continuous spans.

A superimposed dead load of 20 psf is to be included in all designs which account for a possible future concrete overlay wearing surface. The future wearing surface shall be applied between the faces of curbs or parapets and shall be equally distributed among all the girders in the cross section.

For a cross section without a sidewalk, any curb or parapet dead load is distributed equally to all girders.

For a cross section with a sidewalk and barrier on the overhang, sidewalk and barrier dead loads shall be applied to the exterior girder by the lever rule. These loads shall also be applied to the interior girder by dividing the weight equally among all the girders. A more detailed discussion of dead load distribution can be found in 17.2.8.

19.3.3.4 Live Load

The HL-93 live load shall be used for all new bridges. Refer to section 17.2.4.2 for a detailed description of the HL-93 live load, including the design truck, design tandem, design lane, and double truck.

19.3.3.5 Live Load Distribution

The live load distribution factors shall be computed as specified in **LRFD [4.6.2.2].** Table 17.2-7 summarizes the equations required for prestressed I-girders. The moment and shear distribution factors for prestressed I-girders are determined using equations that consider girder spacing, span length, deck thickness, the number of girders, skew and the longitudinal stiffness parameter. See the WisDOT policy item for live load distribution factors for prestressed box girders.

Separate shear and moment distribution factors are computed for interior and exterior girders. The applicability ranges of the distribution factors shall also be considered. If the applicability ranges are not satisfied, then conservative assumptions must be made based on sound engineering judgment.

WisDOT policy item:

The typical cross section for prestressed box girders shall be type “g” as illustrated in **LRFD [Table 4.6.2.2.1-1]**.

For prestressed box girders, the St. Venant torsional inertia, J, may be calculated as closed thin-walled sections for sections with voids, and as solid sections for sections without voids in accordance with **LRFD [C4.6.2.2.1]**.

See 17.2.8 for additional information regarding live load distribution.

19.3.3.6 Dynamic Load Allowance

The dynamic load allowance, IM, is given by **LRFD [3.6.2]**. Dynamic load allowance equals 33% for all live load limit states except the fatigue limit state and is not applied to pedestrian loads or the lane load portion of the HL-93 live load. See 17.2.4.3 for further information regarding dynamic load allowance.

19.3.3.7 Prestressed I-Girder Deck Design

The design of concrete decks on prestressed I-girders is based on **LRFD [4.6.2.1]**. Moments from truck wheel loads are distributed over a width of deck which spans perpendicular to the girders. This width is known as the distribution width and is given by **LRFD [Table 4.6.2.1.3-1]**. See 17.5 for further information regarding deck design.

19.3.3.8 Composite Section

The effective flange width is the width of the deck slab that is to be taken as effective in composite action for determining resistance for all limit states. The effective flange width, in accordance with **LRFD [4.6.2.6]**, is equal to the tributary width of the girder for interior girders. For exterior girders, it is equal to one half the effective flange width of the adjacent interior girder plus the overhang width. The effective flange width shall be determined for both interior and exterior beams.

Since the deck concrete has a lower strength than the girder concrete, it also has a lower modulus of elasticity. Therefore, when computing composite section properties, the effective flange width (as stated above) must be reduced by the ratio of the modulus of elasticity of the deck concrete divided by the modulus of elasticity of the girder concrete.

WisDOT exception to AASHTO:

WisDOT uses the formulas shown below to determine E_c for prestressed girder design. For 6 ksi girder concrete, E_c is 5,500 ksi, and for 4 ksi deck concrete, E_c is 4,125 ksi. The E_c value of 5,500 ksi for 6 ksi girder concrete strength was determined from deflection studies. These equations are used in place of those presented in **LRFD [5.4.2.4]** for the following calculations: strength, section properties, and deflections due to externally applied dead and live loads.

For slab concrete strength other than 4 ksi, E_c is calculated from the following formula:

$$E_c = \frac{4{,}125\sqrt{f'_c}}{\sqrt{4}} \text{ (ksi)}$$

For girder concrete strengths other than 6 ksi, E_c is calculated from the following formula:

$$E_c = \frac{5{,}500\sqrt{f'_c}}{\sqrt{6}} \text{ (ksi)}$$

WisDOT policy item:

WisDOT uses the equation presented in **LRFD [5.4.2.4]** (and shown below) to calculate the modulus of elasticity at the time of release using the specified value of f'ci. This value of E_i is used for loss calculations and for girder camber due to prestress forces and girder self weight.

$$E_c = 33{,}000 \cdot K_1 \cdot w_c^{1.5}\sqrt{f'_{ci}}$$

Where:

K_1	=	Correction factor for source of aggregate, use 1.0 unless previously approved by BOS.
w_c	=	Unit weight of concrete, 0.150 (kcf)
f'_{ci}	=	Specified compressive strength of concrete at the time of release (ksi)

19.3.3.9 Design Stress

In many cases, stress at the Service III limit state in the bottom fiber at or near midspan after losses will control the flexural design. Determine a trial strand pattern for this condition and proceed with the flexural design, adjusting the strand pattern if necessary.

The design stress is the sum of the Service III limit state bottom fiber stresses due to non-composite dead load on the basic girder section, plus live load, dynamic load allowance and superimposed dead load on the composite section, as follows:

$$f_{des} = \frac{M_{d(nc)}}{S_{b(nc)}} + \frac{M_{d(c)} + M_{(LL+IM)}}{S_{b(c)}}$$

Where:

f_{des}	=	Service III design stress at section (ksi)
$M_{d(nc)}$	=	Service III non-composite dead load moment at section (k-in)
$M_{d(c)}$	=	Service III superimposed dead load moment at section (k-in)
$M_{(LL+IM)}$	=	Service III live load plus dynamic load allowance moment at section (k-in)
$S_{b(nc)}$	=	Non-composite section modulus for bottom of basic beam (in^3)
$S_{b(c)}$	=	Composite section modulus for bottom of basic beam (in^3)

The point of maximum stress is generally 0.5 of the span for both end and intermediate spans. But for longer spans (over 100'), the 0.4 point of the end span may control and should be checked.

19.3.3.10 Prestress Force

With f_{des} known, compute the required effective stress in the prestressing steel after losses, f_{pe}, needed to counteract all the design stress except an amount of tension equal to the tensile stress limit listed in **LRFD [Table 5.9.2.3.2b-1]**. The top of the girder is subjected to severe corrosion conditions and the bottom of the girder is subjected to moderate exposure. The Service III tensile stress at the bottom fiber after losses for pretensioned concrete shall not exceed $0.19\lambda\sqrt{f'_c}$ (or 0.6 ksi) ; where λ = concrete density modification factor **LRFD [5.4.2.8]**, and has a value of 1.0 for normal weight concrete. Therefore:

$$f_{pe} = f_{des} - \min(0.19\sqrt{f'_c} \text{ or } 0.6 \text{ ksi})$$

Note: A conservative approach used in hand calculations is to assume that the allowable tensile stress equals zero.

Applying the theory discussed in 19.2:

$$f_{pe} = \frac{P_{pe}}{A}\left(1 + \frac{ey}{r^2}\right)$$

Where:

P_{pe}	=	Effective prestress force after losses (kips)
A	=	Basic beam area (in^2)
e	=	Eccentricity of prestressing strands with respect to the centroid of the basic beam at section (in)
r	=	$\sqrt{\frac{I}{A}}$ of the basic beam (in)

For prestressed box girders, assume an e and apply this to the above equation to determine P_{pe} and the approximate number of strands. Then a trial strand pattern is established using the Standard Details as a guide, and a check is made on the assumed eccentricity. For prestressed I-girders, f_{pe} is solved for several predetermined patterns and is tabulated in the Standard Details.

Present practice is to detail all spans of equal length with the same number of strands, unless a span requires more than three additional strands. In this case, the different strand arrangements are detailed along with a plan note stating: "The manufacturer may furnish all girders with the greater number of strands."

19.3.3.11 Service Limit State

Several checks need to be performed at the service limit state. Refer to the previous narrative in 19.3.3 for sections to be investigated and section 17.2.3.2 for discussion on the service limit state. Note that Service I limit state is used when checking compressive stresses and Service III limit state is used when checking tensile stresses.

The following should be verified by the engineer:

- Verify that the Service III tensile stress due to beam self-weight and prestress applied to the basic beam at transfer does not exceed the limits presented in **LRFD [Table 5.9.2.3.1b-1]**, which depend upon whether or not the strands are bonded and satisfy stress requirements. This will generally control at the top of the beam near the beam ends where the dead load moment approaches zero and is not able to counter the tensile stress at the top of the beam induced by the prestress force. When the calculated tensile stress exceeds the stress limits, the strand pattern must be modified by draping or partially debonding the strand configuration.

- Verify that the Service I compressive stress due to beam self-weight and prestress applied to the basic beam at transfer does not exceed 0.65 f'_{ci}, as presented in **LRFD [5.9.2.3.1a]**. This will generally control at the bottom of the beam near the beam ends or at the hold-down point if using draped strands.

- Verify that the Service III tensile stress due to all dead and live loads applied to the appropriate sections after losses does not exceed the limits presented in **LRFD [Table 5.9.2.3.2b-1]**. No tensile stress shall be permitted for unbonded strands. The tensile stress of bonded strands shall not exceed $0.19\lambda\sqrt{f'_c}$ (or 0.6 ksi) as all strands shall be considered to be in moderate corrosive conditions. This will generally control at the bottom of the beam near midspan and at the top of the continuous end of the beam. The value of λ is 1.0 for normal weight concrete **LRFD [5.4.2.8]**.

- Verify that the Service I compressive stress due to all dead and live loads applied to the appropriate sections after losses does not exceed the limits presented in **LRFD [Table 5.9.2.3.2a-1]**. Two checks need to be made for girder bridges. The compressive stress due to the sum of effective prestress and permanent loads shall not exceed 0.45 f'_c (ksi). The compressive stress due to the sum of effective prestress, permanent loads and transient loads shall not exceed $0.60\phi_w f'_c$ (ksi). The term ϕ_w, a reduction factor applied to thin-walled box girders, shall be 1.0 for WisDOT standard girders.

- Verify that Fatigue I compressive stress due to fatigue live load and one-half the sum of effective prestress and permanent loads does not exceed 0.40 f'_c (ksi) **LRFD [5.5.3.1]**.

- Verify that the Service I compressive stress at the top of the deck due to all dead and live loads applied to the appropriate sections after losses does not exceed 0.40 f'_c.

WisDOT policy item:

The top of the prestressed I-girders at interior supports shall be designed as reinforced concrete members at the strength limit state in accordance with **LRFD [5.12.3.3.6]**. In this case, the stress limits for the service limit state shall not apply to this region of the precast girder.

19.3.3.12 Raised, Draped or Partially Debonded Strands

When straight strands are bonded for the full length of a prestressed girder, the tensile and compressive stresses near the ends of the girder will likely exceed the allowable service limit state stresses. This occurs because the strand pattern is designed for stresses at or near midspan, where the dead load moment is highest and best able to balance the effects of the prestress. Near the ends of the girder this dead load moment approaches zero and is less able to balance the prestress force. This results in tensile stresses in the top of the girder and compressive stresses in the bottom of the girder. The allowable initial tensile and compressive stresses are presented in the first two bullet points of 19.3.3.11. These stresses are a function of f'_{ci}, the compressive strength of concrete at the time of prestress force transfer. Transfer and development lengths should be considered when checking stresses near the ends of the girder.

The designer should start with a straight (raised), fully bonded strand pattern. If this overstresses the girder near the ends, the following methods shall be utilized to bring the girder within the allowable stresses. These methods are listed in order of preference and discussed in the following sections:

1. Use raised strand pattern (If excessive top flange reinforcement or if four or more additional strands versus a draped strand pattern are required, consider the draped strand alternative)

2. Use draped strand pattern

3. Use partially debonded strand pattern (to be used sparingly)

Only show one strand pattern per span (i.e. Do not show both raised and draped span alternatives for a given span).

A different girder spacing may need to be selected. It is often more economical to add an extra girder line than to maximize the number of strands and use debonding.

Prestressed box girders strands are to be straight, bonded, and located as shown in the Standard Details.

19.3.3.12.1 Raised Strand Patterns

Some of the standard strand patterns listed in the Standard Details show a raised strand pattern. Generally strands are placed so that the center of gravity of the strand pattern is as close as possible to the bottom of the girder. With a raised strand pattern, the center of gravity of the strand pattern is raised slightly and is a constant distance from the bottom of the girder for its entire length. Present practice is to show a standard raised arrangement as a preferred alternate to draping for short spans. For longer spans, debonding at the ends of the strands is an alternate (see 19.3.3.12.3). Use 0.6" strands for all raised patterns.

19.3.3.12.2 Draped Strand Patterns

Draping some of the strands is another available method to decrease stresses from prestress at the ends of the I-beam where the stress due to applied loads are minimum.

The typical strand profile for this technique is shown in Figure 19.3-1.

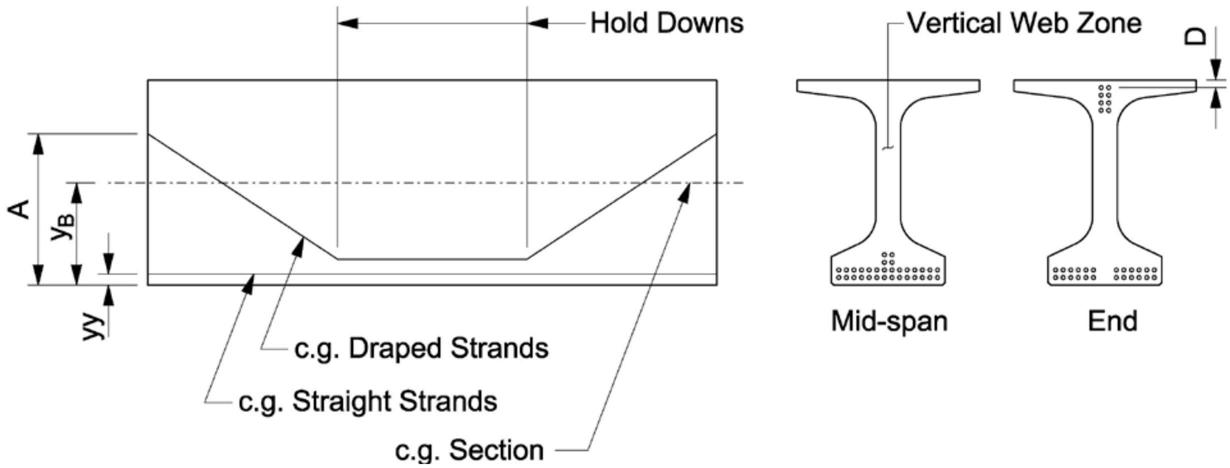

Figure 19.3-1
Typical Draped Strand Profile

Note that all the strands that lie within the "vertical web zone" of the mid-span arrangement are used in the draped group.

The engineer should show only one strand size for the draped pattern on the plans. Use only 0.5" strands for the draped pattern on 28" and 36" prestressed I-girders and 0.6" strands for all raised (straight) patterns for these shapes. Use 0.6" strands, only, for 36W", 45W", 54W", 72W" and 82W" prestressed I-girders. See Chapter 40 standards for 45", 54" and 70" prestressed I-girders.

Hold-down points for draped strands are located approximately between the 1/3 point and the 4/10 point from each end of the girder. The Standard Details, Prestressed Girder Details, show B values at the 1/4 point of the girder. On the plan sheets provide values for B_{min} and B_{max} as determined by the formulas shown on the Standards.

The maximum slope specified for draped strands is 12%. This limit is determined from the safe uplift load per strand of commercially available strand restraining devices used for hold-downs. The minimum distance, D, allowed from center of strands to top of flange is 2". For most designs, the maximum allowable slope of 12% will determine the location of the draped strands. Using a maximum slope will also have a positive effect on shear forces.

Initial girder stresses are checked at the end of the transfer length, which is located 60 strand diameters from the girder end. The transfer length is the embedment length required to develop f_{pe}, the effective prestressing steel stress (ksi) after losses. The prestressing steel stress varies linearly from 0.0 to f_{pe} along the transfer length.

The longer full development length of the strand is required to reach the larger prestressing steel stress at nominal resistance, f_{ps} (ksi). The strand stress is assumed to increase linearly from f_{pe} to f_{ps} over the distance between the transfer length and development length.

Per **LRFD [5.9.4.3.2]**, the development length is:

$$\ell_d \geq \kappa\left(f_{ps} - \frac{2}{3}f_{pe}\right)d_b$$

Where:

d_b	=	Nominal strand diameter (in)
κ	=	1.0 for members with a depth less than or equal to 24”, and 1.6 for members with a depth of greater than 24”

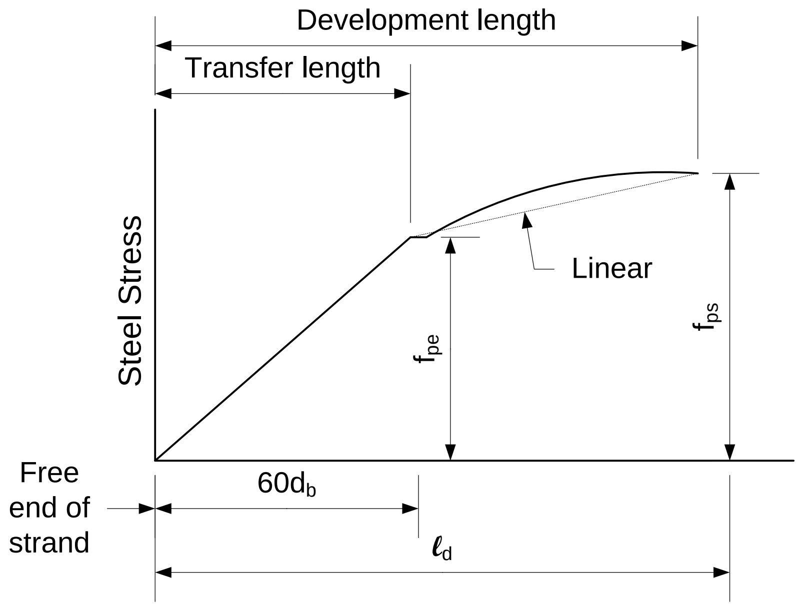

Figure 19.3-2
Transfer and Development Length

19.3.3.12.3 Partially Debonded Strand Patterns

The designer may use debonded strands if a raised or draped strand configuration fails to meet the allowable service stresses. The designer should exercise caution when using debonded strands as this may not result in the most economical design. Partially debonded strands are fabricated by wrapping sleeves around individual strands for a specified length from the ends of the girder, rendering the bond between the strand and the girder concrete ineffective for the wrapped, or shielded, length.

Bond breakers should only be applied to interior strands as girder cracking has occurred when they were applied to exterior strands. In computing bond breaker lengths, consideration is given to the theoretical stresses at the ends of the girder. These stresses are due entirely to prestress. As a result, the designer may compute a stress reduction based on certain strands having bond breakers. This reduction can be applied along the length of the debonded strands.

Partially debonded strands must adhere to the requirements listed in **LRFD [5.9.4.3.3]**. The list of requirements is as follows:

- The development length of partially debonded strands shall be calculated in accordance with **LRFD [5.9.4.3.2]** with $\kappa = 2.0$.
- The number of debonded strands shall not exceed 25% of the total number of strands.
- The number of debonded strands in any horizontal row shall not exceed 40% of the strands in that row.
- The length of debonding shall be such that all limit states are satisfied with consideration of the total developed resistance (transfer and development length) at any section being investigated.
- Not more than 40% of the debonded strands, or four strands, whichever is greater, shall have debonding terminated at any section.
- The strand pattern shall be symmetrical about the vertical axis of the girder. The consideration of symmetry shall include not only the strands being debonded but their debonded length as well, with the goal of keeping the center of gravity of the prestress force at the vertical centerline of the girder at any section. If the center of gravity of the prestress force deviates from the vertical centerline of the girder, the girder will twist, which is undesirable.
- Exterior strands in each horizontal row shall be fully bonded for crack control purposes.

19.3.3.13 Strength Limit State

The design factored positive moment is determined using the following equation:

$$M_u = 1.25DC + 1.50DW + 1.75(LL + IM)$$

The Strength I limit state is applied to both simple and continuous span structures. See 17.2.4 for further information regarding loads and load combinations.

19.3.3.13.1 Factored Flexural Resistance

The nominal flexural resistance assuming rectangular behavior is given by **LRFD [5.6.3.2.3]** and **LRFD [5.6.3.2.2]**.

The section will act as a rectangular section as long as the depth of the equivalent stress block, a, is less than or equal to the depth of the compression flange (the structural deck thickness). Per **LRFD [5.6.3.2.2]**:

$$a = c\beta_1$$

Where:

c	=	Distance from extreme compression fiber to the neutral axis assuming the tendon prestressing steel has yielded (in)
β_1	=	Stress block factor **LRFD [5.6.2.2]**

By neglecting the area of mild compression and tension reinforcement, the equation presented in **LRFD [5.7.3.1.1]** for rectangular section behavior reduces to:

$$c = \frac{A_{ps} f_{pu}}{\alpha_1 f'_c \beta_1 b + k A_{ps} \frac{f_{pu}}{d_p}}$$

Where:

A_{ps}	=	Area of prestressing steel (in^2)
f_{pu}	=	Specified tensile strength of prestressing steel (ksi)
f'_c	=	Compressive strength of the flange ($f'_{c(deck)}$ for rectangular section) (ksi)
b	=	Width of compression flange (in)
k	=	0.28 for low relaxation strand per **LRFD [C5.6.3.1.1]**
d_p	=	Distance from extreme compression fiber to the centroid of the prestressing tendons (in)
α_1	=	Stress block factor; equals 0.85 (for $f'_C \leq 10.0$ ksi) **LRFD [5.6.2.2]**

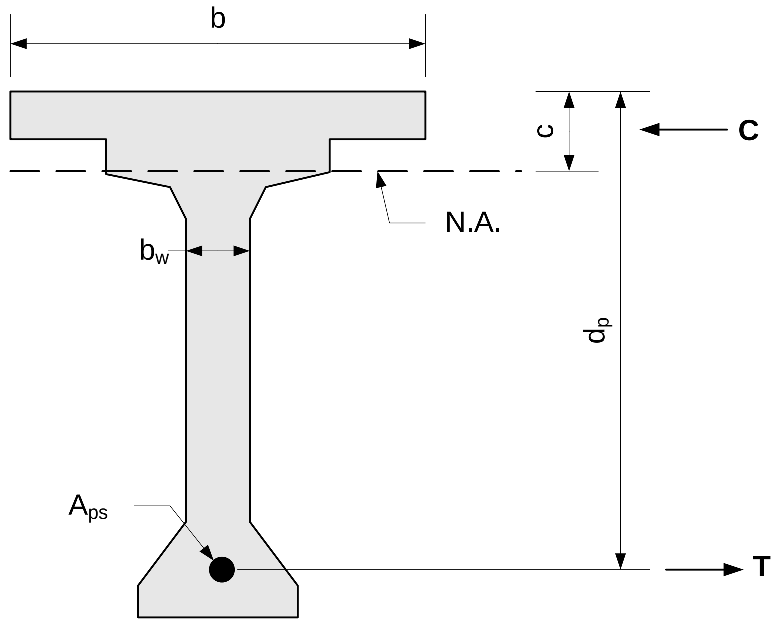

Figure 19.3-3
Depth to Neutral Axis, c

Verify that rectangular section behavior is allowed by checking that the depth of the equivalent stress block, a, is less than or equal to the structural deck thickness. If it is not, then T-section behavior provisions should be followed. If the T-section provisions are used, the compression block will be composed of two different materials with different compressive strengths. In this situation, **LRFD [C5.6.2.2]** recommends using β_1 and α_1 corresponding to the lower f'_c. The following equation for c shall be used for T-section behavior: **LRFD [5.6.3.1.1]**

$$c = \frac{A_{ps} f_{pu} - \alpha_1 f'_c (b - b_w) h_f}{\alpha_1 f'_c \beta_1 b_w + k A_{ps} \dfrac{f_{pu}}{d_p}}$$

Where:

b_w	=	Width of web (in) – use the top flange width if the compression block does not extend below the haunch.
h_f	=	Depth of compression flange (in)

The factored flexural resistance presented in **LRFD [5.6.3.2.2]** is simplified by neglecting the area of mild compression and tension reinforcement. Furthermore, if rectangular section behavior is allowed, then $b_w = b$, where b_w is the web width as shown in Figure 19.3-3. The equation then reduces to:

$$M_r = \phi A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right)$$

Where:

M_r	=	Factored flexural resistance (kip-in)
ϕ	=	Resistance factor
f_{ps}	=	Average stress in prestressing steel at nominal bending resistance (refer to **LRFD [5.6.3.1.1]**) (ksi)

If the T-section provisions must be used, the factored moment resistance equation is then:

$$M_r = \phi A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + \alpha_1 \phi f'_c (b - b_w) h_f \left(\frac{a}{2} - \frac{h_f}{2}\right)$$

Where:

h_f	=	Depth of compression flange with width, b (in)

The engineer must then verify that M_r is greater than or equal to M_u.

<u>WisDOT exception to AASHTO:</u>

WisDOT standard prestressed I-girders and strand patterns are tension-controlled. The ε_t check, as specified in **LRFD [5.6.2.1]**, is not required when the standard girders and strand patterns are used, and $\phi = 1$.

19.3.3.13.2 Minimum Reinforcement

Per **LRFD [5.6.3.3]**, the minimum amount of prestressed reinforcement provided shall be adequate to develop a M_r at least equal to the lesser of M_{cr}, or $1.33M_u$.

M_{cr} is the cracking moment, and is given by:

$$M_{cr} = \gamma_3 \left[S_c (\gamma_1 f_r + \gamma_2 f_{cpe}) - 12M_{dnc} [(S_c/S_{nc}) - 1] \right]$$

Where:

S_c	=	Section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (in^3)
f_r	=	Modulus of rupture (ksi)

f_{cpe}	=	Compressive stress in concrete due to effective prestress forces only (after losses) at extreme fiber of section where tensile stress is caused by externally applied loads (ksi)
M_{dnc}	=	Total unfactored dead load moment acting on the basic beam (k-ft)
S_{nc}	=	Section modulus for the extreme fiber of the basic beam where tensile stress is caused by externally applied loads (in^3)
γ_1	=	1.6 flexural cracking variability factor
γ_2	=	1.1 prestress variability factor
γ_3	=	1.0 for prestressed concrete structures

Per **LRFD [5.4.2.6]**, the modulus of rupture for normal weight concrete is given by:

$f_r = 0.24\lambda\sqrt{f'_c}$; where λ = conc. density modification factor **LRFD [5.4.2.8]**, and has a value of 1.0 for normal weight conc.

19.3.3.14 Non-prestressed Reinforcement

Non-prestressed reinforcement consists of bar steel reinforcement used in the conventional manner. It is placed longitudinally along the top of the member to carry any tension which may develop after transfer of prestress. The designer should completely detail all rebar layouts including stirrups.

The amount of reinforcement is that which is sufficient to resist the total tension force in the concrete based on the assumption of an uncracked section.

For draped designs, the control is at the hold-down point of the girder. At the hold-down point, the initial prestress is acting together with the girder dead load stress. This is where tension due to prestress is still maximum and compression due to girder dead load is decreasing.

For non-draped designs, the control is at the end of the member where prestress tension exists but dead load stress does not.

Note that a minimum amount of reinforcement is specified in the Standards. This is intended to help prevent serious damage due to unforeseeable causes like improper handling or storing.

19.3.3.15 Horizontal Shear Reinforcement

The horizontal shear reinforcement resists the Strength I limit state horizontal shear that develops at the interface of the slab and girder in a composite section. The dead load used to calculate the horizontal shear should only consider the DC and DW dead loads that act on the composite section. See 17.2.4 for further information regarding the treatment of dead loads and load combinations.

$$V_u = 1.25DC + 1.50DW + 1.75(LL + IM)$$

$$V_{ni} \geq V_{ui} / \phi$$

Where:

V_u	=	Maximum strength limit state vertical shear (kips)
V_{ui}	=	Strength limit state horizontal shear at the girder/slab interface (kips)
V_{ni}	=	Nominal interface shear resistance (kips)
ϕ	=	0.90 per **LRFD [5.5.4.2]**

The shear stress at the interface between the slab and the girder is given by:

$$v_{ui} = \frac{V_u}{b_{vi} d_v}$$

Where:

v_{ui}	=	Factored shear stress at the slab/girder interface (ksi)
b_{vi}	=	Interface width to be considered in shear transfer (in)
d_v	=	Distance between the centroid of the girder tension steel and the mid-thickness of the slab (in)

The factored horizontal interface shear shall then be determined as:

$$V_{ui} = 12 v_{ui} b_{vi}$$

The nominal interface shear resistance shall be taken as:

$$V_{ni} = cA_{cv} + \mu \left[A_{vf} f_y + P_c \right]$$

Where:

A_{cv}	=	Concrete area considered to be engaged in interface shear transfer. This value shall be set equal to $12b_{vi}$ (ksi)
c	=	Cohesion factor specified in **LRFD [5.7.4.4]**. This value shall be taken as 0.28 ksi for WisDOT standard girders with a cast-in-place deck

μ	=	Friction factor specified in **LRFD [5.7.4.4]**. This value shall be taken as 1.0 for WisDOT standard girders with a cast-in-place deck (dim.)
A_{vf}	=	Area of interface shear reinforcement crossing the shear plan within the area A_{cv} (in^2)
f_y	=	Yield stress of shear interface reinforcement not to exceed 60 (ksi)
P_c	=	Permanent net compressive force normal to the shear plane (kips)

P_c shall include the weight of the deck, haunch, parapets, and future wearing surface. A conservative assumption that may be considered is to set $P_c = 0.0$.

The nominal interface shear resistance, V_{ni}, shall not exceed the lesser of:

$$V_{ni} \le K_1 f'_c A_{cv} \quad \text{or} \quad V_{ni} \le K_2 A_{cv}$$

Where:

K_1	=	Fraction of concrete strength available to resist interface shear as specified in **LRFD [5.7.4.4]**. This value shall be taken as 0.3 for WisDOT standard girders with a cast-in-place deck (dim.)
K_2	=	Limiting interface shear resistance as specified in **LRFD [5.7.4.4]**. This value shall be taken as 1.8 ksi for WisDOT standard girders with a cast-in-place deck

WisDOT policy item:

The stirrups that extend into the deck slab presented on the Standards are considered adequate to satisfy the minimum reinforcement requirements of **LRFD [5.7.4.2]**

19.3.3.16 Web Shear Reinforcement

Web shear reinforcement consists of placing conventional reinforcement perpendicular to the axis of the girder.

WisDOT policy item:

Web shear reinforcement shall be designed by **LRFD [5.7.3.4.2]** (General Procedure) using the Strength I limit state for WisDOT standard girders.

WisDOT prefers girders with spacing symmetrical about the midspan in order to simplify design and fabrication. The designer is encouraged to simplify the stirrup arrangement as much as possible. For vertical stirrups, the required area of web shear reinforcement is given by the following equation:

$$A_v \geq \frac{(V_n - V_c)s}{f_y d_v \cot\theta} \quad \text{(or } 0.0316\lambda\sqrt{f'_c}\frac{b_v s}{f_y} \text{ minimum , } \textbf{LRFD [5.7.2.5]}\text{)}$$

Where:

A_v	=	Area of transverse reinforcement within distance, s (in^2)
V_n	=	Nominal shear resistance (kips)
V_c	=	Nominal shear resistance provided by tensile stress in the concrete (kips)
s	=	Spacing of transverse reinforcement (in)
f_y	=	Specified minimum yield strength of transverse reinforcement (ksi)
d_v	=	Effective shear depth as determined in **LRFD [5.7.2.8]** (in)
b_v	=	Minimum web width within depth, d_v
λ	=	Concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

$\cot\theta$ shall be taken as follows:

- When $V_{ci} < V_{cw}$, $\cot\theta = 1.0$
- When $V_{ci} > V_{cw}$, $\cot\theta = 1.0 + 3\left(\frac{f_{pc}}{\sqrt{f'_c}}\right) \leq 1.8$

$$V_u = 1.25DC + 1.5DW + 1.75(LL + IM)$$
$$V_n = V_u / \phi$$

Where:

V_u	=	Strength I Limit State shear force (kips)
ϕ	=	0.90 per **LRFD [5.5.4.2]**

See 17.2 for further information regarding load combinations.

Per **LRFD [5.7.3.3]**, determine V_c as given by:

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

Where:

β = Factor indicating ability of diagonally cracked concrete to transmit tension and shear. **LRFD [5.7.3.4]**

λ = Concrete density modification factor; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

$\beta = \frac{4.8}{(1+750\varepsilon_s)}$ (For sections containing at least the minimum amount of transverse reinforcement specified in **LRFD [5.7.2.5]**)

Where:

ε_s = Net longitudinal tensile strain in the section at the centroid of the tension reinforcement.

$$= \frac{\left(\frac{|M_u|}{d_v}+0.5N_u+|V_u-V_p|-A_{ps}f_{po}\right)}{E_sA_s+E_pA_{ps}}$$

Where:

$|M_u|$ = Absolute value of the factored moment at the section, not taken less than $|V_u-V_p|d_v$ (kip-in.)

N_u = Factored axial force, taken as positive if tensile and negative if compression (kip)

V_p = Componet of prestressing force in the direction of the shear force; positive if resisiting the applied shear (kip)

A_{ps} = Area of prestressing steel on the flexural tension side of the member (in^2).

A_s = Area of nonprestressing steel on the flexural tension side of the member (in^2).

f_{po} = A parameter taken as modulus of elasticity of prestressing steel multiplied by the locked-in difference in strain between the prestressing steel and the surrounding concrete (ksi).

WisDOT policy item:

Based on past performance, for prestressed I-girders the upper limit for web reinforcement spacing, s_{max}, per **LRFD [5.7.2.6]** will be reduced to 18 inches.

When determining shear reinforcement, spacing requirements as determined by analysis at 1/10th points, for example, should be carried-out to the next 1/10th point. As an illustration, spacing requirements for the 1/10th point should be carried out to very close to the 2/10th point, as the engineer, without a more refined analysis, does not know what the spacing requirements would be at the 0.19 point. For the relatively small price of stirrups, don't shortchange the shear capacity of the prestressed girder.

The web reinforcement spacing shall not exceed the maximum permitted spacing determined as:

- If $\upsilon_u < 0.125f'_c$, then $s_{max} = 0.8d_v \leq 18"$
- If $\upsilon_u \geq 0.125f'_c$, then $s_{max} = 0.4d_v \leq 12"$

Where:

$$\upsilon_u = \frac{V_u - \phi V_p}{\phi b_v d_v}$$ per **LRFD [5.7.2.8]**.

The nominal shear resistance, $V_c + V_s$, is limited by the following:

$$V_c + \frac{A_v f_y d_v \cot\theta}{s} \leq 0.25f'_c b_v d_v$$

Reinforcement in the form of vertical stirrups is required at the extreme ends of the girder. The stirrups are designed to resist 4% of the total prestressing force at transfer at a unit stress of 20 ksi and are placed within h/4 of the girder end, where h is the total girder depth. For a distance of 1.5d from the ends of the beams, reinforcement shall be placed to confine the prestressing steel in the bottom flange. The reinforcement shall be shaped to enclose the strands, shall be a #3 bar or greater and shall be spaced at less than or equal to 6”. Note that the reinforcement shown on the Standard Details sheets satisfies these requirements.

Welded wire fabric may be used for the vertical reinforcement. It must be deformed wire with a minimum size of D18.

Per **LRFD [5.7.3.5]**, at the inside edge of the bearing area to the section of critical shear, the longitudinal reinforcement on the flexural tension side of the member shall satisfy:

$$A_s f_y + A_{ps} f_{ps} \geq \left(\frac{V_u}{\phi} - 0.5V_s \right) \cot\theta$$

In the above equation, $\cot\theta$ is as defined in the V_c discussion above, and V_s is the shear reinforcement resistance at the section considered. Any lack of full reinforcement development shall be accounted for. Note that the reinforcement shown on the Standard Detail sheets satisfies these requirements.

19.3.3.17 Continuity Reinforcement

The design of non-prestressed reinforcement for negative moment at the support is based on the Strength I limit state requirements of **LRFD [5.6.3]**:

$$M_u = 1.25DC + 1.50DW + 1.75(LL + IM)$$

LRFD [5.5.4.2] allows a ϕ factor equal to 0.9 for tension-controlled reinforced concrete sections such as the bridge deck.

The continuity reinforcement consists of mild steel reinforcement in the deck in the negative moment region over the pier. Consider both the non-composite and the superimposed dead loads and live loads for the Strength I design of the continuity reinforcement in the deck.

Moment resistance is developed in the same manner as shown in 19.3.3.13.1 for positive moments, except that the bottom girder flange is in compression and the deck is in tension. The moment resistance is formed by the couple resulting from the compression force in the bottom flange and the tension force from the longitudinal deck steel. Consider A_s to consist of the longitudinal deck steel present in the deck slab effective flange width as determined in 19.3.3.8. The distance, d_p, is taken from the bottom of the girder flange to the center of the longitudinal deck steel.

WisDOT exception to AASHTO:

Composite sections formed by WisDOT standard prestressed I-girders shall be considered to be tension-controlled for the design of the continuity reinforcement. The ε_t check, as specified in **LRFD [5.6.2.1]**, is not required, and $\phi = 0.9$.

WisDOT policy item:

New bridge designs shall consider only the top mat of longitudinal deck steel when computing the continuity reinforcement capacity.

WisDOT policy item:

The continuity reinforcement shall be based on the greater of either the interior girder design or exterior girder and detailed as typical reinforcement for the entire width of the bridge deck. However, do not design the continuity steel based on the exterior girder design beneath a raised sidewalk. The continuity steel beneath a raised sidewalk should not be used for rating.

Based on the location of the neutral axis, the bottom flange compressive force may behave as either a rectangle or a T-section. On WisDOT standard prestressed I-girders, if the depth of the compression block, a, falls within the varying width of the bottom flange, the compression block acts as an idealized T-section. In this case, the width, b, shall be taken as the bottom flange width, and the width, b_w, shall be taken as the bottom flange width at the depth “a”. During T-section behavior, the depth, h_f, shall be taken as the depth of the bottom flange of full width, b. See Figure 19.3-4 for details. Ensure that the deck steel is adequate to satisfy $M_r \geq M_u$.

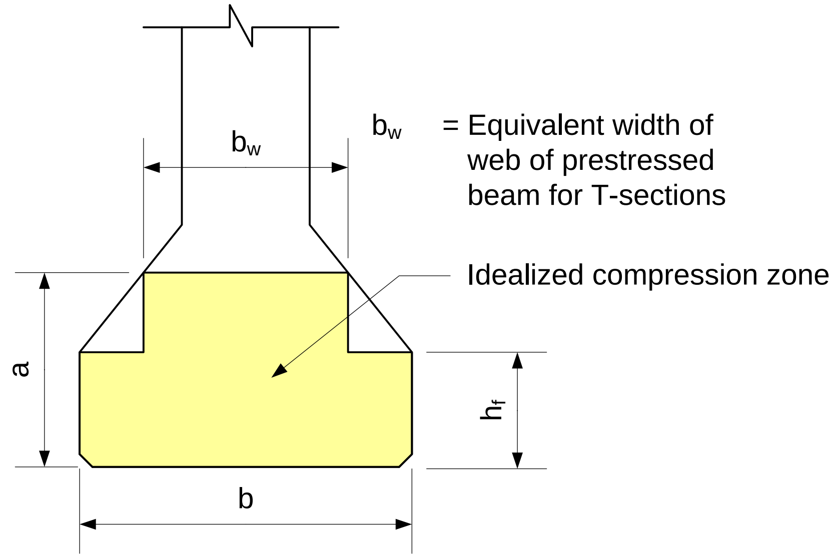

Figure 19.3-4
T-Section Compression Flange Behavior

The continuity reinforcement should also be checked to ensure that it meets the crack control provisions of **LRFD [5.6.7]**. This check shall be performed assuming severe exposure conditions. Only the superimposed loads shall be considered for the Service and Fatigue requirements.

The concrete between the abutting girder ends is usually of a much lesser strength than that of the girders. However, tests[1] have shown that, due to lateral confinement of the diaphragm concrete, the girder itself fails in ultimate negative compression rather than failure in the material between its ends. Therefore the ultimate compressive stress, f'_c, of the girder concrete is used in place of that of the diaphragm concrete.

This assumption has only a slight effect on the computed amount of reinforcement, but it has a significant effect on keeping the compression force within the bottom flange.

The continuity reinforcement shall conform to the Fatigue provisions of **LRFD [5.5.3]**.

The transverse spacing of the continuity reinforcement is usually taken as the whole or fractional spacing of the D bars as given in 17.5.3.2. Grade 60 bar steel is used for continuity reinforcement. Required development lengths for deformed bars are given in Chapter 9 – Materials.

WisDOT exception to AASHTO:

The continuity reinforcement is not required to be anchored in regions of the slab that are in compression at the strength limit state as stated in **LRFD [5.12.3.3]**. The following locations shall be used as the cut off points for the continuity reinforcement:

1. When ½ the bars satisfy the Strength I moment envelope (considering both the non-composite and composite loads) as well as the Service and Fatigue moment envelopes (considering only the composite moments), terminate ½ of the bars. Extend these bars past this cutoff point a distance not less than the girder depth or 1/16 the clear span for embedment length requirements.

2. Terminate the remaining one-half of the bars an embedment length beyond the point of inflection. The inflection point shall be located by placing a 1 klf load on the composite structure. This cut-off point shall be at least 1/20 of the span length or 4' from point 1, whichever is greater.

Certain secondary features result when spans are made continuous. That is, positive moments develop over piers due to creep[5], shrinkage and the effects of live load and dynamic load allowance in remote spans. The latter only exists for bridges with three or more spans.

These positive moments are somewhat counteracted by negative moments resulting from differential shrinkage[4] between the cast-in-place deck and precast girders along with negative moments due to superimposed dead loads. However, recent field observations cited in **LRFD [C5.12.3.3.2]** suggest that these moments are less than predicted by analysis. Therefore, negative moments caused by differential shrinkage should be ignored in design.

WisDOT exception to AASHTO:

WisDOT requires the use of a negative moment connection only. The details for a positive moment connection per **LRFD [5.12.3.3]** are not compatible with the Standard Details and should not be provided.

19.3.3.18 Camber and Deflection

The prestress camber and dead load deflection are used to establish the vertical position of the deck forms with respect to the girder. The theory presented in the following sections apply to a narrow set of circumstances. The designer is responsible for ensuring that the theoretical camber accounts for the loads applied to the girder. For example, if the diaphragms of a prestressed I-girder are configured so there is one at each of the third points instead of one at midspan, the term in the equation for $\Delta_{nc(DL)}$ related to the diaphragms in 19.3.3.18.2 would need to be modified to account for two point loads applied at the third points instead of one point load applied at midspan.

Deflection effects due to individual loads may be calculated separately and superimposed, as shown in this section. The *PCI Design Handbook* provides design aids to assist the designer in the evaluation of camber and deflection, including cambers for prestress forces and loads, and beam design equations and diagrams.

Figure 19.3-5 illustrates a typical prestressed I-girder with a draped strand profile.

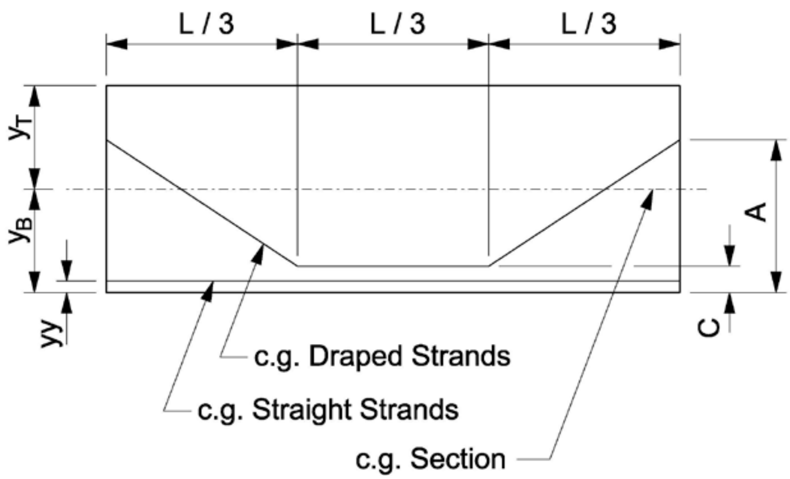

Figure 19.3-5

Typical Draped Strand Profile

19.3.3.18.1 Prestress Camber

The prestressing strands produce moments in the girder as a result of their eccentricity and draped pattern. These moments induce a camber in the girder. The values of the camber are calculated as follows:

Eccentric straight strands induce a constant moment of:

$$M_1 = \frac{1}{12}\left(P_i^s (y_B - yy)\right)$$

Where:

M_1	=	Moment due to initial prestress force in the straight strands minus the elastic shortening loss (k-ft)
P_i^s	=	Initial prestress force in the straight strands minus the elastic shortening loss (kips)
y_B	=	Distance from center of gravity of beam to bottom of beam (in)
yy	=	Distance from center of gravity of straight strands to bottom of beam (in)

This moment produces an upward deflection at midspan which is given by:

$$\Delta_s = \frac{M_1 L^2}{8E_i I_b} \quad \text{(with all units in inches and kips)}$$

For moments expressed in kip-feet and span lengths expressed in feet, this equation becomes the following:

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For the strength limit state, the minimum number of required shear connectors, n, is computed for a given region according to the following equation:

$$n = \frac{P}{Q_r}$$

Where:

P = Total nominal shear force determined as specified in **LRFD [6.10.10.4.2]** (kips)

Q_r = Factored shear resistance of one shear connector (kips)

The given regions over which the required number of shear connectors is distributed are defined based on the point of maximum moment due to live load plus dynamic load allowance. This value is used because it applies to the composite section and is easier to locate than a maximum of the sum of all the moments acting on the composite section.

In most cases, the connector spacing (using three connectors per row) based on fatigue requirements is more than adequate for the strength design requirements. However for relatively long spans, additional shear connectors may be required to satisfy the strength design requirements.

In addition to the above general requirements, special shear connector requirements at points of permanent load contraflexure are presented in **LRFD [6.10.10.3]**.

Additional information and equations used for LRFD design of shear connectors are presented in **LRFD [6.10.10]**. In addition, a design example for shear connectors is also provided in this *Bridge Manual*.

24.7.6 Continuity Reinforcement

For continuous steel girder bridges, continuity reinforcement in the concrete deck must be considered in regions of negative flexure, as specified in **LRFD [6.10.1.7]**. Continuity reinforcement consisting of small bars with close spacing is intended to control concrete deck cracking.

If the longitudinal tensile stress in the concrete deck due to either the factored construction loads or the Service II load combination exceeds ϕf_r, then the following continuity reinforcement requirements must be satisfied:

- The total cross-sectional area of the longitudinal reinforcement in the deck shall be greater than or equal to one percent of the total cross-sectional area of the concrete deck.

- The required reinforcement shall be placed in two layers uniformly distributed across the deck width, with two-thirds being in the top layer and one-third in the bottom layer.

- The specified minimum yield strength, fy, of the reinforcing steel shall not be less than 60 ksi.

- The size of the reinforcement bars shall not exceed No. 6 bars.

- The spacing of the reinforcement bars shall not exceed 12 inches.

Tables 17.5-3 and 17.5-4 meet the criteria specified above.

In computing ϕf_r, f_r shall be taken as the modulus of rupture of the concrete (see **LRFD [5.4.2.6]**) and ϕ shall be taken as 0.90, which is the appropriate resistance factor for concrete in tension (see **LRFD [5.5.4.2]**). The longitudinal stresses in the concrete deck are computed as specified in **LRFD [6.10.1.1.1d]**. Superimposed dead loads and live loads are considered to be resisted by the composite section using the short-term modular ratio, n. Non-composite dead loads are supported by the girders alone.

Terminate the continuity reinforcement at the point of non-composite dead load contraflexure plus a development length. The bars are lapped to No. 4 bars.

For non-composite slabs in the negative moment region (on rehabilitation projects), extend the longitudinal reinforcement in Tables 17.5-3 and 17.5-4 a development length beyond the first shear connectors.

24.8 Field Splices

24.8.1 Location of Field Splices

Field splices shall be placed at the following locations whenever it is practical:

- At or near a point of dead load contraflexure for continuous spans
- Such that the maximum rolling length of the flange plates is not exceeded, thus eliminating one or more butt splices
- At a point where the fatigue in the net section of the base metal is minimized
- Such that section lengths between splices are limited to 120', unless special conditions govern

24.8.2 Splice Material

For homogeneous girders, the splice material is the same as the members being spliced. Generally, 3/4" diameter high-strength A325 bolted friction-type connectors, conforming to ASTM F3125, are used unless the proportions of the structure warrant larger diameter bolts.

24.8.3 Design

The following is a general description of the basic steps required for field splice design. These procedures and the accompanying equations are described in greater detail in **LRFD [6.13.6]**.

24.8.3.1 Obtain Design Criteria

The first design step is to identify the appropriate design criteria. This includes defining material properties, identifying relevant superstructure information and determining the splice location based on the criteria presented in 24.8.1.

Resistance factors used for field splice design are as presented in 17.2.6.

When calculating the nominal slip resistance of a bolt in a slip-critical connection, the value of the surface condition factor, K_s, shall be taken as follows:

- For structures that are to be painted, use Ks = 0.33.
- For unpainted, blast-cleaned steel or steel with organic zinc paint, use Ks = 0.50.

Where a section changes at a splice, the smaller of the two connected sections should be used in the design, as specified in **LRFD [6.13.6.1.1]**.

24.8.3.1.1 Section Properties Used to Compute Stresses

The section properties used to compute stresses are described in **LRFD [6.10.1.1.1]**.

For calculating flexural stresses in sections subjected to positive flexure, the composite sections for short-term (transient) and long-term (permanent) moments shall be based on n and 3n, respectively.

For calculating flexural stresses in sections subjected to negative flexure, the composite section for both short-term and long-term moments shall consist of the steel section and the longitudinal reinforcement within the effective width of the concrete deck, except as specified otherwise in **LRFD [6.6.1.2.1]**, **LRFD [6.10.1.1.1d]** or **LRFD [6.10.4.2.1]**.

WisDOT policy item:

When computing composite section properties based on the steel section and the longitudinal reinforcement within the effective width of the concrete deck, only the top layer of reinforcement shall be considered.

Where moments due to short-term and long-term loads are of opposite sign at the strength limit state, the associated composite section may be used with each of these moments if the resulting net stress in the concrete deck due to the sum of the factored moments is compressive. Otherwise, the provisions of **LRFD [6.10.1.1.1c]** shall be used to determine the stresses in the steel section. Stresses in the concrete deck shall be determined as specified in **LRFD [6.10.1.1.1d]**.

However, for members with shear connectors provided throughout their entire length that also satisfy the provisions of **LRFD [6.10.1.7]**:

- Flexural stresses caused by Service II loads applied to the composite section may be computed using the short-term or long-term composite section, as appropriate, assuming the concrete deck is effective for both positive and negative flexure, as described in **LRFD [6.10.4.2.1]**.

- Live load stresses and stress ranges for fatigue design may be computed using the short-term composite section assuming the concrete deck to be effective for both positive and negative flexure, as described in **LRFD [6.6.1.2.1]**.

WisDOT policy item:

When stresses at the top and bottom of the web are required for web splice design, the flange stresses at the mid-thickness of the flanges can be conservatively used. This allows use of the same stress values for both the flange and web splices, which simplifies the calculations.

24.8.3.1.2 Constructability

As described in **LRFD [6.13.6.1.4a]**, splice connections shall be proportioned to prevent slip during the erection of the steel and during the casting of the concrete deck.

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27.1 General

Bridges supported in the conventional way by abutments and piers require bearings to transfer girder reactions without overstressing the supports, ensuring that the bridge functions as intended. Bridges usually require bearings that are more elaborate than those required for building columns, girders and trusses. Bridge bearings require greater consideration in minimizing forces caused by temperature change, friction and restraint against elastic deformations. A more detailed analysis in bridge bearing design considers the following:

- Bridges are usually supported by reinforced concrete substructure units, and the magnitude of the horizontal thrust determines the size of the substructure units. The coefficient of friction on bridge bearings should be as low as possible.

- Bridge bearings must be capable of withstanding and transferring dynamic forces and the resulting vibrations without causing eventual wear and destruction of the substructure units.

- Most bridges are exposed to the elements of nature. Bridge bearings are subjected to more frequent and greater total expansion and contraction movement due to changes in temperature than those required by buildings. Since bridge bearings are exposed to the weather, they are designed as maintenance-free as possible.

WisDOT policy item:

The temperature range considered for steel girder superstructures is -30°F to 120°F. A temperature setting table for steel bearings is used for steel girders; where 45°F is the neutral temperature, resulting in a range of 120° - 45° = 75° for bearing design.

The temperature range considered for prestressed concrete girder superstructures is 5°F to 85°F. Using an installation temperature of 60° for prestressed girders, the resulting range is 60° - 5° = 55° for bearing design. For prestressed girders an additional shrinkage factor of 0.0003 ft/ft shall also be accounted for. No temperature setting table is used for prestressed concrete girders.

See the Standard for Steel Expansion Bearing Details to determine bearing plate “A” sizing (steel girders) or anchor plate sizing (prestressed concrete girders). This standard also gives an example of a temperature setting table for steel bearings when used for steel girders.

WisDOT policy item:

According to **LRFD [14.4.1]**, the influence of dynamic load allowance need not be included for bearings. However, dynamic load allowance shall be included when designing bearings for bridges in Wisconsin. Apply dynamic load allowance in **LRFD [3.6.2]** to HL-93 live loads as stated in **LRFD [3.6.1.2, 3.6.1.3]** and distribute these loads, along with dead loads, to the bearings.

The compressive deflection, δ, of the bearing shall be limited to ensure the serviceability of the deck joints, seals and other components of the bridge. Deflections of elastomeric bearings due to total load and to live load alone should be considered separately. Relative deflections across joints must be restricted so that a step doesn't occur at a deck joint. **LRFD [C14.7.5.3.6]** recommends that a maximum relative live load deflection across a joint be limited to 1/8".

WisDOT policy item:

WisDOT uses a live load + creep deflection limit of 1/8" for elastomeric bearing design.

Laminated (steel reinforced) elastomeric bearings have a nonlinear load deflection curve in compression. In the absence of information specific to the particular elastomer to be used, **LRFD [Figure C14.7.6.3.3-1]** may be used as a guide. Creep effects should be determined from information specific to the elastomeric compound used. Use the material properties given in this section. The compressive deflection, δ, can be determined as specified in **LRFD [14.7.5.3.6, 14.7.6.3.3]** and by the following equation:

$$\delta = \sum \varepsilon_i \, h_{ri}$$

Where:

δ	=	Instantaneous deflection (inches)
ε_i	=	Instantaneous compressive strain in the i^{th} elastomer layer of a laminated (steel reinforced) bearing
h_{ri}	=	Thickness of i^{th} elastomeric layer in a laminated (steel reinforced) bearing (inches)

Based on **LRFD [14.7.6.3.3]**, the initial compressive deflection of a plain elastomeric pad or in any layer of a laminated (steel reinforced) elastomeric bearing at the service limit state without dynamic load allowance shall not exceed $0.09h_{ri}$.

8. Check anchorage

WisDOT exception to AASHTO:

Design anchorage for laminated elastomeric bearings if the unfactored dead load stress is less than 200 psi. This is an exception to **LRFD [14.8.3]** based on past practice and good performance of existing bearings.

The factored force due to the deformation of an elastomeric element shall be taken as specified in **LRFD [14.6.3.1]** by the following equation:

$$H_u > GA\frac{\Delta_u}{h_{rt}}$$

Where:

H_u	=	Lateral load from applicable strength load combinations in **LRFD [Table 3.4.1-1]** (kips)
G	=	Shear modulus of the elastomer (ksi)
A	=	Plan area of elastomeric element or bearing (inches^2)
Δ_u	=	Factored shear deformation (inches)
h_{rt}	=	Total elastomer thickness (inches)

9. Check reinforcement **LRFD [14.7.5.3.5, 14.7.6.3.7]**

Reinforcing steel plates increase compressive and rotational stiffness, while maintaining flexibility in shear. The reinforcement must have adequate capacity to handle the tensile stresses produced in the plates as they counter the lateral bulging of the elastomer layers due to compression. These tensile stresses increase with compressive load. The reinforcement thickness must also satisfy the requirements of the *AASHTO LRFD Bridge Construction Specifications*. The reinforcing steel plates can be checked as specified in **LRFD [Equation 14.7.5.3.5-1,2]**:

$$h_s \geq \frac{3\, h_{max}\, \sigma_s}{F_y} \text{ for service limit state}$$

$$h_s \geq \frac{2.0\, h_{max}\, \sigma_L}{\Delta F_{TH}} \text{ for fatigue limit state}$$

Where:

h_s	=	Thickness of the steel reinforcement (inches)
h_{max}	=	Thickness of the thickest elastomeric layer in elastomeric bearing (inches)
σ_s	=	Service average compressive stress due to total load (ksi)
Fy	=	Yield strength of steel reinforcement (ksi)
σ_L	=	Service average compressive stress due to live load (ksi)
ΔF_{TH}	=	Constant amplitude fatigue threshold for Category A as specified in **LRFD [Table 6.6.1.2.5-3]** (ksi)

If holes exist in the reinforcement, the minimum thickness shall be increased by a factor equal to twice the gross width divided by the net width.

5. Design the guides and restraints, if applicable **LRFD [14.7.9]**

6. Design the PTFE sliding surface, if applicable **LRFD [14.7.2]**

7. Design the sole plate, masonry plate (or bearing plate), anchorage and connections for pot bearings; design the sole plate, masonry plate (or bearing plate), upper and lower plates, anchorage and connections for disc bearings; as applicable **LRFD [Section 6, 14.8 and 14.7.8.5]**

8. Check the concrete or steel support **LRFD [5.6.5 and Section 6]**

Although the steps for pot and/or disc bearing design are given above, the actual bearing design is typically done by the manufacturer. The design of the masonry plate is done either by the design engineer or by the bearing manufacturer (this should be coordinated and noted in the contract documents).

When using pot or disc bearings, the design plans need to specify the following:

- Degree of fixity (fixed, guided in one direction or non-guided)
- Maximum vertical load
- Minimum vertical load
- Maximum horizontal load (fixed and guided, only)
- Assumed bearing height

Note: The loads specified shall be Service I limit state loads, including dynamic load allowance.

Field adjustments to the given beam seat elevations will be required if the actual bearing height differs from the assumed bearing height stated on the plan. To facilitate such an adjustment without affecting the structural integrity of the substructure unit, a concrete pedestal (plinth) is detailed at each bearing location. Detailing a pedestal height of 10” based on the assumed bearing height will give sufficient room for adjustment should the actual bearing height differ from the assumed bearing height.

27.3 Hold Down Devices

Hold down devices are additional elements added to the Type "A-T" bearings for situations where live load can cause uplift at the abutment end of a girder. Ideally, proper span configurations would eliminate the need for hold down devices as they have proven to be a maintenance problem. Details for hold down devices are given in the Standard for Hold Down Devices.

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E27-1 DESIGN EXAMPLE - STEEL REINFORCED ELASTOMERIC BEARING

ThIs design example is for a 3-span prestressed girder structure. The piers are fixed supports and the abutments accommodate expansion.
(Example is current through LRFD Eighth Edition - 2017)

E27-1.1 Design Data

Bearing location: Abutment (Type A3)

Girder type: 72W

$L_{exp} := 220$ Expansion length, ft

$b_f := 2.5$ Bottom flange width, ft

$DL_{serv} := 167$ Service I limit state dead load, kips

$DL_{ws} := 23$ Service I limit state future wearing surface dead load, kips

$LL_{serv} := 62$ Service I limit state live load, kips

$h_{rcover} := 0.25$ Elastomer cover thickness, in

$h_s := 0.125$ Steel reinforcement thickness, in

$F_y := 36$ Minimum yield strength of the steel reinforcement, ksi

Temperature Zone:	D (Use for Entire State)	**LRFD [Fig. 14.7.5.2-1]**
Minimum Grade of Elastomer:	4	**LRFD [Table 14.7.5.2-1]**
Elastic Hardness:	Durometer 60 +/- 5	(used 55 for design)
Shear Modulus (G):	0.1125 ksi < G <0.165 ksi	**LRFD [Table 14.7.6.2-1]**
Creep Deflection @ 25 Years		
divided by instantaneous deflection:	0.3	**LRFD [Table 14.7.6.2-1]**

E27-1.2 Design Method

Use Design Method A **LRFD [14.7.6]**
Method A results in a bearing with a lower capacity than a bearing designed using Method B. However the increased capacity resulting from the use of Method B requires additional testing and quality control.

E27-1.3 Dynamic Load Allowance

The influence of impact need not be included for bearings **LRFD [14.4.1]**; however, dynamic load allowance will be included to follow a **WisDOT policy item**.

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28.1 Introduction

Many structures have joints that must be properly designed and installed to insure their integrity and serviceability. Bridges as well as highway pavements, airstrips, buildings, etc. need joints to take care of expansion and contraction caused by temperature changes. However, bridges expand and contract more than pavement slabs or buildings and have their own special types of expansion devices.

Current practice is to limit the number of bridge expansion joints. This practice results in more movement at each joint. There are so many potential problems associated with joints that fewer joints are recommended practice. Expansion joints are placed on the high end of a bridge if only one joint is placed on the bridge. This is done to prevent the bridge from creeping downhill and to minimize the amount of water passing over the joint.

Open joints generally lead to future maintenance. Water and debris fall through the joint. Water running through an open joint erodes the soil under the structure, stains the bent cap and columns, and leads to corrosion of adjacent girders, diaphragms, and bearings. During freeze-thaw conditions, large icicles may form under the structure or ice may form on the roadway presenting a traffic hazard. Debris acts with water in staining the substructure units and plugs the drainage systems.

In the past, open steel finger type joints were used on long span bridges where large movements encountered. Finger joints were placed in the span near the point of contraflexure and were placed on the structure where they are required structurally. Drains were located to prevent drainage across the joint if feasible. In some areas, they were provided with a drainage trough to collect the water passing through.

Sliding steel plate joints are semi-open joints since water and light debris can pass through. A sealant placed in the joint prevents some water from passing through. It also prevents the accumulation of debris which can keep the joint from moving as it was designed. To date, considerable maintenance has occurred with sealants and neoprene troughs have been added to collect the water at some sites.

Currently finger and sliding plate details are maintained for joint maintenance and retrofitting but are not used for new structures. Watertight expansion devices such as strip seals and modular types are recommended for new structures. Although these expansion joints are not completely watertight; they have been effective in reducing damage to adjacent girders, diaphragms, bearings and substructure units.

The neoprene compression seal is a closed joint which is watertight if it is properly installed and an adequate adhesive is employed. Compression seals are only used for fixed joints. Strip-seals are watertight joints which are used in place of sliding plate joints or finger joints in an attempt to keep water and debris on the bridge deck surface.

Refer to Figure 12.7-1 for placement of expansion devices. The following criteria is used for placement of expansion devices:

28.2 Compression Seals

28.2.1 Description

This is a preformed, compartmented, elastomeric polychloropene (neoprene) device. In the past, compression seals were used sparingly on fixed joints provided there was little or no movement of the joint. However, compression seals shall no longer be used in this application due to the fact that the seals tend to leak over time. Compression seals shall be used only in longitudinal construction joint locations or for rehabilitation projects that do not involve full joint replacement (i.e., where the existing seal has pulled out of the joint and needs to be replaced).

28.2.2 Joint Design

Most applications have been for bridge rehabilitation where the seal is installed into the concrete deck without armor.

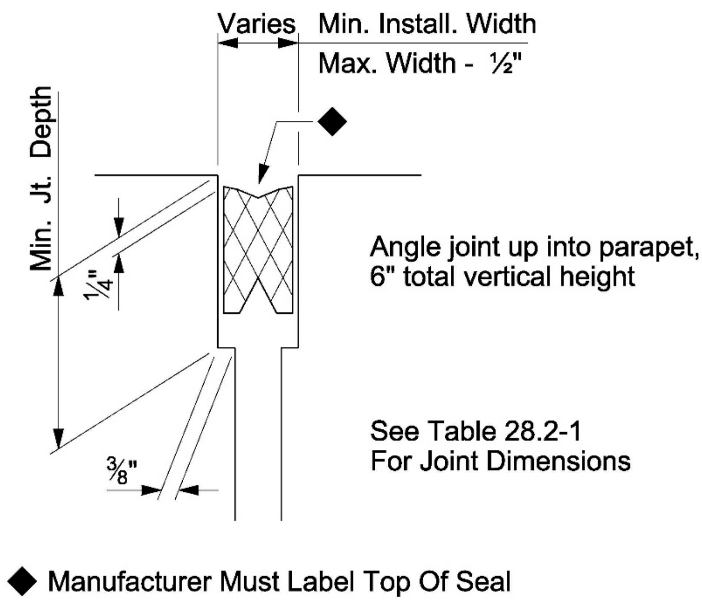

Figure 28.2-1 .
Joint Design

Manufacturer must label top of seal.

28.2.3 Seal Size

The width of the compression seal to be used in a given joint opening is computed by adding the as-constructed joint width plus a small width safety factor. For best results oversize the seal by a minimum of ½ inch. See Table 28.2-1 for approximate dimensions.

28.2.4 Installation

Ease of installation is achieved using a lubricant-adhesive which as the name implies acts initially as a lubricant then cures out to form an adhesive membrane between the contact faces of the angle and seal. This membrane, together with the compressive action of the seal, is designed to provide a waterproof joint interface.

The following information is a guide for the installation of neoprene compressive seals:

1. Thorough cleaning of joint faces is essential. Forced air or manual dusting handles most cases; use a solvent on oily areas.

2. Require application of the manufacturer's lubricant-adhesive to the sides of the neoprene seal as well as the joint faces. An adequate coating of the lubricant-adhesive is helpful in installation.

3. Proper installation tools consist of hand or machine tools that compress and eject the seal or weighted rollers that squeeze it in place. Screwdrivers, pry bars or other sharp ended tools which may puncture the seal are not allowed.

4. Stretching in excess of 5% is not permitted.

5. It is imperative that the seal be installed below the pavement surface. The minimum depth recess to top of seal is ¼ inch. Turn joint up into parapet at an angle, 6 inches total height.

6. Prior to shipping, all compression seals are to be labeled TOP SIDE by the manufacturers. Field installation reports indicate difficulty in determining TOP SIDE for some types of seals. Also, the seal cross-section is not shown on a shop drawing unless the joint is armored.

28.2.5 Maintenance

Manual removal of incompressible materials which tend to collect within the joint opening is desirable. However, in most cases this is not necessary since the tire forces the material against the elastic neoprene seal which rebounds causing the material to bounce up and out of the seal.

It is essential to the operation of the seal that no form of hot or cold joint filler be placed over the top of the seal. This includes resurfacing mats or overlays. The reasons are as follows:

1. Hot fillers may either melt the seal or seriously affect the elastomeric properties for future performance.

2. The filler acts as a constant media of transmitting undue vertical tire forces to the compression seal which may break the interface bond.

SEAL WIDTH	SEAL HEIGHT	MIN. JOINT WIDTH	MAX. JOINT WIDTH	*MIN. INSTALL. WIDTH	MIN. JOINT DEPTH
2	2 ±	1 ±	1 ¾ ±	1 ¼ ±	2 ⅞ ±
2 ¼	2 ½ ±	1 ±	2 ±	1 ⅜ ±	3 ⅛±
2 ½	2 ¾ ±	1 ¼ ±	2 ¼ ±	1 ⅝ ±	3 ⅝ ±
3	3 ⅜ ±	1 ⅜ ±	2 ½ ±	1 ¾ ±	4 ¼ ±
3 ½	3 ½ ±	1 ½ ±	3 ±	2 ⅛ ±	4 ¾ ±
4	4 ½ ±	1 ¾ ±	3 ½ ±	2 ⅝ ±	5 ½ ±

Table 28.2-1
Approximate Compression Seal Dimensions

* This is the minimum practical limit as suggested by the seal manufacturer.

28.3 Strip Seal Expansion Devices

28.3.1 Description

Strip seal expansion devices are molded neoprene glands inserted and mechanically locked between armored interfaces of extruded steel sections. The name "Strip Seal" is derived from the strip profile of the neoprene seal. During structure movements a preformed central hinge enables the strip seal profile to fold between the seal extrusions. Strip seal design details are given on the Standards for Strip Seal Expansion Joint Details and Strip Seal Cover Plate Details.

Ease of installation is attained by applying a lubricant-adhesive to the steel extrusions; which as the name implies acts initially as a lubricant; then cures to form an adhesive membrane between contact surfaces of the extrusions and neoprene gland. The neoprene glands are generally inserted in the field by using a tire-iron type tool. A minimum transverse roadway surface opening between the extrusions of 1 ½ inches or greater will facilitate the field installation of the neoprene gland. When extra size or travel capacity is available, joint openings can be increased to facilitate gland installation keeping the maximum transverse roadway joint opening at 4 inches for new construction. Greater openings may be used on maintenance projects only.

The strip seal is readily adaptable to changes in interfacial elevations as well as longitudinal skew deformations. The neoprene gland is installed as one continuous length on any given joint application. Additional considerations are given to the "racking" movement on the neoprene gland as the structure skew angle increases.

28.3.2 Curb and Parapet Sections

The strip seal is curved up into the curb or safety parapet with cover plates. The details are shown on Standard for Strip Seal Cover Plate Details. The resulting recess between the parapet and joint requires cover plates for maintenance considerations.

28.3.3 Median and Sidewalk Sections

Median cover plates are not required if the joint is placed at the median surface, otherwise they are required. All sidewalk joints must have cover plates as shown on the standard details.

28.3.4 Size Selection

The first consideration in strip seal size selection is the effective expansion length for the given joint location. Table 28.1-1 is established in accordance with AASHTO Specifications by employing a cold climate temperature range given in 28.1.4 for determining the maximum span lengths for the joint movement limits. The span length was decreased for prestressed girder structures to further accommodate movements due to concrete creep and shrinkage. The “Maximum Expansion Length” for a given joint size and structure type is shown in Table 28.3-1

On new structures and deck replacements, provide details for strip seal models having a minimum size of 4 inches. If the skew angle exceeds 30 degrees, limit the actual racking

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30.1 Crash-Tested Bridge Railings and FHWA Policy

Notice: All contracts with a letting date after December 31, 2019 must use bridge rails and transitions meeting the 2016 Edition of MASH criteria for new permanent installation and full replacement.

WisDOT policy item:

For all Interstate structures, the 42SS parapet shall be used. For all STH and USH structures with a posted speed >= 45 mph, the 42SS parapet shall be used.

The timeline for implementation of the above policy is:

- All contracts with a letting date after December 31, 2019.
  (This is an absolute, regardless of when the design was started.)

- All preliminary designs starting after October 1, 2017
  (Even if the let is anticipated to be prior to December 31, 2019.)

Contact BOS should the 42" height adversely affect sight distance, a minimum 0.5% grade for drainage cannot be achieved, or for other non-typical situations.

Crash test procedures for full-scale testing of guardrails were first published in 1962 in the *Highway Research Correlation Services Circular 482*. This was a one-page document that specified vehicle mass, impact speed, and approach angle for crash testing and was aimed at creating uniformity to traffic barrier research between several national research agencies.

In 1974, the National Cooperative Highway Research Program (NCHRP) published their final report based on NCHRP Project 22-2, which was initiated to address outstanding questions that were not covered in *Circular 482*. The final report, NCHRP Report 153 – *"Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances,"* was widely accepted following publication; however, it was recognized that periodic updating would be required.

NCHRP Project 22-2(4) was initiated in 1979 to address major changes to reflect current technologies of that time and NCHRP Report 230, *"Recommended Procedures for the Safety Performance Evaluation of Highway Safety Appurtenances,"* was published in 1980. This document became the primary reference for full-scale crash testing of highway safety appurtenances in the U.S. through 1993.

In 1986, the Federal Highway Administration (FHWA) issued a policy memorandum that stated highway bridges on the National Highway System (NHS) and the Interstate Highway System (IHS) must use crash-tested railings in order to receive federal funding.

The American Association of State Highway and Transportation Officials (AASHTO) recognized that the evolution of roadside safety concepts, technology, and practices necessitated an update to NCHRP Report 230 approximately 7 years after its adoption. NCHRP Report 350, *"Recommended Procedures for the Safety Performance Evaluation of Highway Features,"* represented a major update to the previously adopted report. The updates

The application of bridge railings shall comply with the following guidance:

1. All bridge railings shall conform to **MASH 2016 requirements for lets after December 31, 2019**.

2. Traffic Railings placed on state-owned and maintained structures (Interstate Highways, United States Highways, State Trunk Highways, and roadways over such highways) with a design speed exceeding 45 mph shall be solid concrete parapets. Where the minimum 0.5% deck grade cannot be accommodated for proper drainage based on project specific constraints, the designer shall utilize open railings as described in this section. (**NOTE**: WisDOT does not currently have an open rail meeting the minimum MASH TL-3 requirements for NHS roadways or non-NHS roadways with design speeds exceeding 45 mph. An open rail meeting MASH TL-3 is being investigated.).

   Traffic Railings placed on locally-owned and maintained structures (County Trunk Highways, Local Roadways) with a design speed exceeding 45 mph are strongly encouraged to utilize solid concrete parapets.

3. Traffic Railings placed on structures with a design speed of 45 mph or less can be either solid concrete parapets or open railings with the exception as noted below in the single slope parapet application section. It should be noted that open railing bridges can incur maintenance issues with salt-water runoff over the edge of deck.

4. New bridge plans utilizing concrete parapets shall be designed with single-sloped ("SS") parapets. See item No. 1 below for usage.

5. Per **LRFD [13.8.1]** and **LRFD [13.9.2]**, the minimum height of a Pedestrian (and/or bicycle) Railing shall be 42" measured from the top of the walkway or riding surface respectively. Per the *Wisconsin Bicycle Facility Design Handbook*, on bridges that are signed or marked as bikeways and bicyclists are operating right next to the railing, the preferred height of the railing is 54". The higher railing/parapet height is especially important and should be used on long bridges, high bridges, and bridges having high bicyclist volumes. If an open railing is used, the clear opening between horizontal elements shall be 6 inches or less.

6. Aesthetics associated with bridge railings shall follow guidance provided in 30.4.

The designation for railing types are shown on the Standard Details. Bridge railings shall be employed as follows:

1. <u>The default parapet shall be the "42SS"</u>. If site distance issues arise due to the 42-inch height, please contact BOS for consideration of a shorter parapet ("32SS"and "36SS"). Single slope parapet "56SS" shall only be used if 56" CBSS adjoins the bridge. The "42SS" is TL-4 under MASH. The "32SS" is TL-3 under MASH. The "36SS" is TL-4 under MASH. *At this time, the "56SS" Test Loading is still unknown.*

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

2. The sloped face parapet "LF" and "HF" parapets shall be used as Traffic Railings for rehabilitation projects (joint repair, impact damage, etc.) only to match the existing parapet type. The sloped face parapets were crash-tested per NCHRP Report 230 and meet NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

3. The "51F" parapet shall only be used as a Traffic Railing on the median side of a structure when it provides a continuation of an approach 51 inch high median barrier.

4. Although the vertical face parapet "A" can be used for all design speeds, Bureau of Structures Development Section approval is required for design speeds exceeding 45mph. The vertical face parapet is recommended for use as a Combination Railing on raised sidewalks or as a Traffic Railing where the design speed is 45 mph or less. If the structure has a raised sidewalk on one side only, a sloped parapet should be used on the side opposite of the sidewalk. For design speeds exceeding 45 mph, at locations where the parapet is protected by a Traffic Railing between the roadway and a sidewalk at grade, the vertical face parapet can be used as a Pedestrian Railing. The vertical face parapet "A" is considered at TL-3 when on a bridge deck and TL-2 when on a raised sidewalk (The structural capacity is TL-3, however the vaulting effect of the sidewalk lowers the rating to TL-2).

5. Aesthetic railings may be used if crash tested according to 30.1 or follow the guidance provided in 30.4. See Chapter 4 – Aesthetics for CSS considerations.

   The Texas style aesthetic parapet, type "TX", can be used as a Traffic/Pedestrian Railing on raised sidewalks on structures with a design speed of 45 mph or less. For design speeds exceeding 45 mph, at locations where the parapet is protected by a Traffic Railing between the roadway and a sidewalk at grade, the type "TX" parapet can be used. The type "TX" parapet is TL-2 under MASH.

6. The type "PF" tubular railing, as shown in the Standard Details of Chapter 40, shall not be used on new bridge plans with a PS&E after 2013. This railing was not allowed on the National Highway System (NHS). The type "PF" railing was used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less.

7. Combination Railings, type "C1" through "C6", are shown in the Standard Details and are approved as aesthetic railings attached to concrete parapets. The aesthetic additions are placed at least 5" from the crash-tested rail face per the Standard Details and have previously been determined to not present a snagging potential. Combination railing, type "3T", without the recessed details on the parapet faces may be used when aesthetic details are not desired or when CSS funding is not available (see Chapter 4 – Aesthetics). These railings can only be used when the design speed is 45 mph or less, or the railing is protected by a Traffic Railing between the roadway and a sidewalk at grade. The crash test criteria of the combination railings are based on the concrete parapets to which they are attached.

8. Chain Link Fence and Tubular Screening, as shown in the Standard Details, may be attached to the top of concrete parapets as part of a Combination Railing or as a Pedestrian Railing attached directly to the deck if protected by a Traffic Railing between

the roadway and a sidewalk at grade. Chain Link Fence, when attached to the top of a concrete Traffic Railing, can be used for design speeds exceeding 45 mph. Due to snagging and breakaway potential of the vertical spindles, Tubular Screening should only be used when the design speed is 45 mph or less, or the screening is protected by a Traffic Railing between the roadway and a sidewalk at grade.

9. Type "H" aluminum or steel railing can be used on top of either vertical face or single slope parapets (“A” or “SS”) as part of a Combination Railing when required for pedestrians and/or bicyclists. For a design speed greater than 45 mph, the single slope parapet is recommended. Per the Standard Specifications, the contractor shall furnish either aluminum railing or steel railing. In general, the bridge plans shall include both options. For a specific project, one option may be required. This may occur when rehabilitating a railing to match an existing railing or when painting of the railing is required (requires steel option). If one option is required, the designer shall place the following note on the railing detail sheet: “Type H (insert railing type) railing shall not be used”. The combination railing is TL-3 under MASH.

10. Timber Railing as shown in the Standard Details is not allowed on the National Highway System (NHS). Timber Railing may be used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less. The Timber Railing has not been rated under MASH.

11. The type "W" railing, as shown in the Standard Details, is not allowed on the National Highway System (NHS). This railing may be used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less. The type “W” railing shall be used on concrete slab structures only. The use of this railing on girder type structures has been discontinued. Generally, type "W" railing is considered when the roadway approach requires standard beam guard and if the structure is 80 feet or less in length. Although the type “W” railing was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-3 (based on a May 1997 FHWA memorandum), FHWA has since restricted its use as indicated above.

12. Type “M” steel railing, as shown in the Standard Details, shall generally be used as a Traffic Railing on all functional classes of Wisconsin structures with a design speed of 45 mph or less. The type “M” railing may be used on roadways with a design speed exceeding 45 mph where the minimum 0.5% deck grade cannot be accommodated for proper drainage based on project specific constraints. The type “M” railing also can be used in place of the type “W” railing when placed on girder type structures as type “W” railings are not allowed for this application. However, the type “M” railing is not allowed for use on prestressed box girder bridges. This railing shall be considered where the Region requests an open railing. The type “M” railing is TL-2 under MASH.

13. Type “NY3/NY4” steel railings, as shown in the Standard Details, shall generally be used as a Traffic Railing on all functional classes of Wisconsin structures with a design speed of 45 mph or less. The type “NY3/NY4” railings may be used on roadways with a design speed exceeding 45 mph where the minimum 0.5% deck grade cannot be accommodated for proper drainage based on project specific constraints. The type “NY3/NY4” railings also can be used in place of the type “W” railing when placed on girder type structures as type “W” railings are not allowed for this application. The type

"NY4" railing may be used on a raised sidewalk where the design speed is 45 mph or less. However, the type "NY" railings are not allowed for use on prestressed box girder bridges. These railings shall be considered where the Region requests an open railing. The type "NY" railings are TL-2 under MASH.

14. The type "F" steel railing, as shown in the Standard Details of Chapter 40, shall not be used on new bridge plans with a PS&E after 2013. It has not been allowed on the National Highway System (NHS) in the past and was used on non-NHS roadways with a design speed of 45 mph or less. Details in Chapter 40 are for informational purposes only.

15. If a box culvert has a Traffic Railing across the structure, then the railing members shall have provisions for a thrie beam connection at the ends of the structure as shown in the *Facilities Development Manual (FDM) Standard Detail Drawings (SDD) 14b20*. Railing is not required on box culverts if the culvert is extended to provide an adequate clear zone as defined in *FDM 11-15-1*. Non-traversable hazards or fixed objects should not be constructed or allowed to remain within the clear zone. When this is not feasible, the use of a Traffic Railing to shield the hazard or obstacle may be warranted. The railing shall be provided only when it is cost effective as defined in *FDM Procedure 11-45-1*.

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

See the *FDM* for additional railing application requirements. See *11-45-1* and *11-45-2* for Traffic Barrier, Crash Cushions, and Roadside Barrier Design Guidance. See *11-35-1*, *Table 1.2* for requirements when barrier wall separation between roadway and sidewalk is necessary.

Table of Contents

36.1 Design Method

36.1.1 Design Requirements

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

36.1.2 Rating Requirements

The current version of *AASHTO Manual for Bridge Evaluation (LRFR)* covers rating of concrete box culverts. Refer to 45.8 for additional guidance on load rating various types of culverts.

36.1.3 Standard Permit Design Check

New structures are also to be checked for strength for the 190 kip Wisconsin Standard Permit Vehicle (Wis-SPV), with a single lane loaded, multiple presence factor equal to 1.0, and a live load factor (γ_{LL}) as shown in Table 45.3-3. See 45.12 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.

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.

Box culverts used to carry water should consider the following items:

- 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.
- A minimum vertical opening of 5 feet is desirable for cleaning purposes.

Pedestrian underpasses should consider the following items:

- The minimum opening for pedestrian underpasses is 8 feet high by 10 feet wide. However, when considering maintenance and emergency vehicles or bicyclists the minimum opening should be 10 feet high by 12 feet wide. For additional guidance refer to the Wisconsin Bicycle Facility Design Handbook and the FDM.
- The entire top and 1 foot below the bottom of the top slab should be waterproofed.
- The top of the bottom slab should be sloped with a 1% normal crown to minimize moisture collecting on the travel path. Additionally, 0.5% to 1% longitudinal slope for drainage is recommended.
- Flared wings are recommended at openings. For long underpasses, lighting systems (recessed lights and skylights) should be considered, as well. For additional guidance on user’s comfort, safety measures, and lighting refer to the Wisconsin Bicycle Facility Design Handbook.

Cattle underpasses should consider the following items:

- The minimum size for cattle underpasses is 6 feet high by 5 feet wide.
- Consider providing a minimum longitudinal slope of 1%, desirable 3%, to allow for flushing, but not so steep that the stock will slip. Slopes steeper than 5% should be avoided.
- For additional guidance refer to the FDM.

Aluminum box culverts are not permitted by the Bureau of Structures.

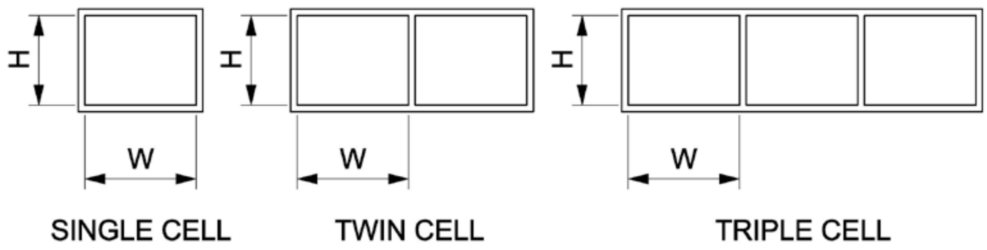


Figure 36.2-1
Typical Cross Sections

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 [C5.4.2.4]**
	=	$(33{,}000)(K_1)(w_C)^{1.5}(f'_C)^{1/2}$ = 3586 ksi

Where:

K_1	=	1.0
W_C	=	0.15 kcf, unit weight of concrete
n	=	Es / Ec = 8, modular ratio **LRFD [5.6.1]**

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.	Strength I Load Factor, γ_{st} 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	1.35	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 [5.6.3.2.3]** (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 [5.6.3.2.1]**, 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 [12.11.5.2]**. 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 [5.12.7.3]**. The location of the critical section for shear for culverts with haunches shall be determined in conformance with **LRFD [C5.12.8.6.1]** 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 [5.12.7.3]** shall be determined as:

$$V_c = \left(0.0676\lambda\sqrt{f'_c} + 4.6\frac{A_s}{bd_e}\frac{V_u d_e}{M_u}\right)bd_e \leq 0.126\lambda\sqrt{f'_c}\,bd_e$$

Where:

$$\frac{V_u d_e}{M_u} \leq 1$$

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.)

λ = Concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

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 [5.7.2.1]** shall be taken as:

$$V_r = \phi V_n = \phi V_c$$

Per **LRFD [5.12.7.3]**, for single-cell box culverts only, Vc for slabs monolithic with walls need not be taken less than:

$$0.0948 \cdot \lambda \sqrt{f'_c}\, b d_e$$

and V_c for slabs simply supported need not be taken less than:

$$0.0791 \cdot \lambda \sqrt{f'_c}\, b d_e$$

The shear resistance of the concrete, V_c, for <u>walls</u> of box culverts with 2.0 feet or more of fill, for one-way action per **LRFD [5.7.3.3]** shall be determined as:

$$V_c = 0.0316 \cdot \beta\lambda \sqrt{f'_c}\, b_v d_v \le 0.25 f'_c\, b_v d_v$$

Where:

V_c = Shear resistance of the concrete (kip)

β = 2.0 (**LRFD [5.7.3.4.1]**)

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 [5.7.2.8]**. Perpendicular distance between tension and compression resultants. Need not be taken less than the greater of $0.9d_e$ or 0.72h (in.)

λ = Concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

36.3.4.3.2 Depth of Fill Less than 2.0 ft

Per **LRFD [5.12.7.3]**, for box culverts with less than 2.0 feet of fill follow **LRFD [5.7]** and **LRFD [5.12.8.6]**.

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 [5.7.3.3]** shall be determined as:

$$V_c = 0.0316 \cdot \beta\lambda\sqrt{f'_c}\, b_v d_v \leq 0.25 f'_c\, b_v d_v$$

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 [12.11.4]**, the provisions of **LRFD [5.6.7]** 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:

$$s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \quad \text{(in.)}$$

Bar spacing, s, need not be less than 5 in. for control of flexural cracking **LRFD [5.6.7]**

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 [5.6.7]** for guidance)

d_c = Thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.). For calculation purposes, d_c, need not be taken greater than 2 in. plus the bar radius

f_{ss} = Tensile stress in steel reinforcement at the service limit state (ksi) $\leq$ 0.6 f_y

h = Overall thickness or depth of the component (in.)

WisDOT Policy Item:

A class 1 exposure factor, γ_e = 1.0, shall be used for all cases for cast-in-place box culverts except for the top steel in the top slab of a box culvert with zero fill, where a class 2 exposure factor, γ_e = 0.75, shall be used.

36.3.6 Minimum Reinforcement Check

Per **LRFD [12.11.5.3]**, the area of reinforcement, A_s, in the box culvert cross-section should be checked for minimum reinforcement requirements per **LRFD [5.6.3.3]**.

The area of tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r, or moment capacity at least equal to the lesser of:

M_{cr} (or) $1.33M_u$

$M_{cr} = \gamma_3 (\gamma_1 f_r) S = 1.1 f_r (I_g / c)$; $S = I_g / c$

Where:

γ_1 = 1.6 flexural cracking variability factor

γ_3 = 0.67 ratio of minimum yield strength to ultimate tensile strength; for A615 Grade 60 reinforcement

f_r = $0.24\lambda\sqrt{f'_c}$ 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)
λ	=	concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

The factored resistance, M_r or moment capacity, shall be calculated as in 36.3.4.2 and shall satisfy:

$$M_r \geq \min(M_{cr} \text{ or } 1.33\, M_u)$$

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 (3.0 times for temperature and shrinkage)
- 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.

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]**.

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)

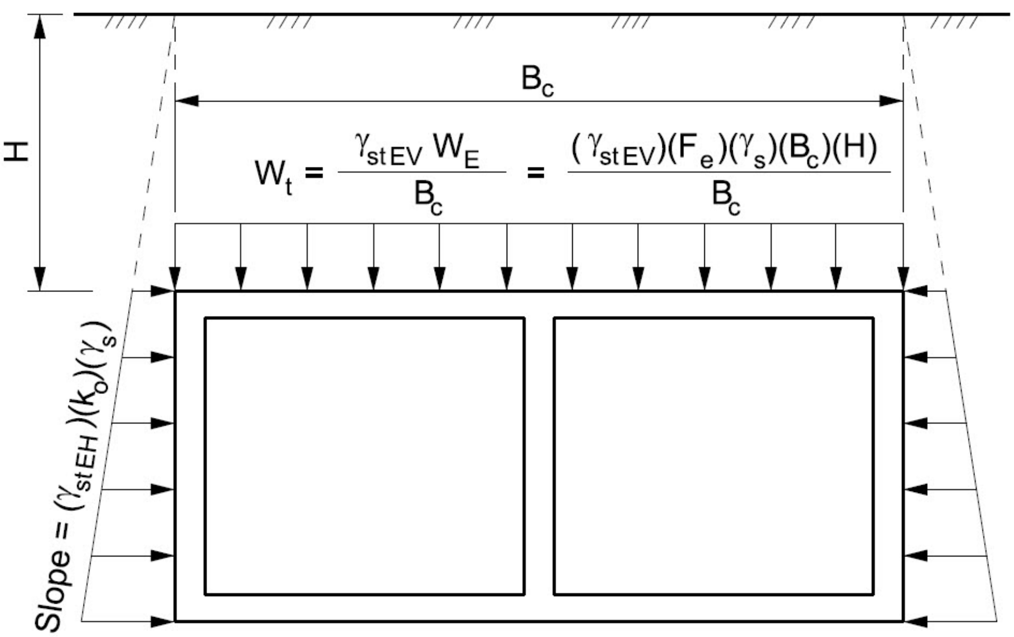


Figure 36.4-1
Factored Vertical and Horizontal Earth Pressures

Where:

W_t = Factored earth pressure on top of box culvert (ksf)

γ_{stEV} = Vertical earth pressure load factor

γ_{stEH} = Horizontal earth pressure load factor

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 soil pressure on the bottom of the box is not shown, but shall be determined for the design of the bottom slab. Note: vertical earth pressures, as well as other loads (e.g. DC and DW), are typically distributed equally over the bottom of the box when determining pressure distributions for the bottom slab. Pressure distributions from a refined analysis is typically not warranted for new culvert designs, but should be considered when evaluating existing culvert sections on culvert extension projects.

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 36.4-1
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.

Where the depth of fill over the box is less than 2 feet, the wheel loads are distributed per **LRFD [4.6.2.10]**. Where the depth of fill is 2 feet or more, the wheel loads shall be uniformly distributed over a rectangular area with sides equal to the dimension of the tire contact area **LRFD [3.6.1.2.5]**, increased by the live load distribution factor (LLDF) in **LRFD [Table 3.6.1.2.6a-1]**, using the provisions of **LRFD [3.6.1.2.6b-c]**. Where areas from distributed wheel loads overlap at the top of the culvert, the total load is considered as uniformly distributed over the rectangular area (A_{LL}) defined by the outside limits described in **LRFD [3.6.1.2.6b-c]**.

Per **LRFD [3.6.1.2.6a]**, for single-span culverts, the effects of live load may be neglected where the depth of fill is more than 8.0 feet and exceeds the span length. For multiple span culverts, the effects may be neglected where the depth of fill exceeds the distance between inside faces of end walls.

Skew is not considered for design loads.

36.4.6.1 Depth of Fill Less than 2.0 ft.

Where the depth of fill is less than 2.0 ft, follow **LRFD [4.6.2.10]**.

36.4.6.1.1 Case 1 – Traffic Travels Parallel to Span

When the traffic travels primarily parallel to the span, follow **LRFD [4.6.2.10.2]**. Use a single lane and the single lane multiple presence factor of 1.2.

Distribution length perpendicular to the span:

$$E = (96 + 1.44(S))$$

Where:

E	=	Equivalent distribution width perpendicular to span (in.)
S	=	Clear span (ft)

The distribution of wheel loads perpendicular to the span for depths of fill less than 2.0 feet is illustrated in Figure 36.4-2.

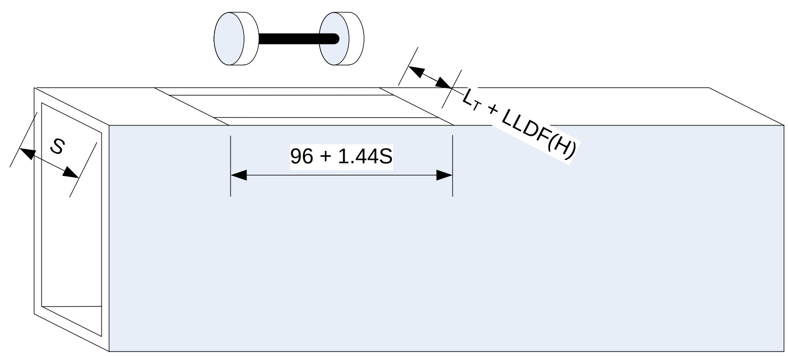


Figure 36.4-2
Distribution of Wheel Loads Perpendicular to Span, Depth of Fill Less than 2.0 feet

Distribution length parallel to the span:

$E_{span} = (L_T + LLDF\ (H))$

Where:

E_{span}	=	Equivalent distribution length parallel to span (in.)
L_T	=	Length of tire contact area parallel to span, as specified in **LRFD [3.6.1.2.5]** (in.)
LLDF	=	Factor for distribution of live load with depth of fill, 1.15, as specified in **LRFD [Table 3.6.1.2.6a-1]**.
H	=	Depth of fill from top of culvert to top of pavement (in.)

The distribution of wheel loads parallel to the span for depths of fill less than 2.0 feet is illustrated in Figure 36.4-3.

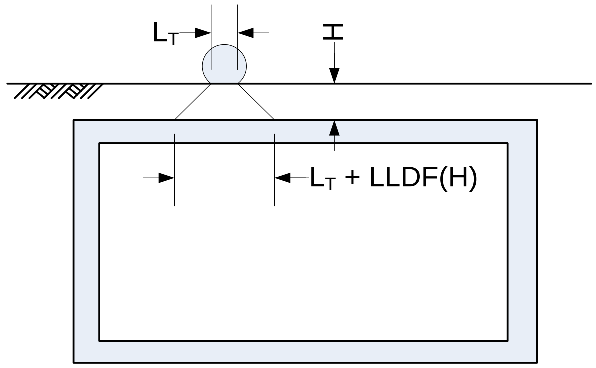


Figure 36.4-3
Distribution of Wheel Loads Parallel to Span, Depth of Fill Less than 2.0 feet

36.4.6.1.2 Case 2 - Traffic Travels Perpendicular to Span

When traffic travels perpendicular to the span, live load shall be distributed to the top slab using the equations specified in **LRFD [4.6.2.1]** for concrete decks with primary strips perpendicular to the direction of traffic per **LRFD[4.6.2.10.3]**. The effect of multiple lanes shall be considered. Use the multiple presence factor, m, as required per **LRFD [3.6.1.1.2]**.

For a cast-in-place box culvert, the width of the primary strip, in inches is:

+M: 26.0 + (6.6)(S)

-M: 48.0 + (3.0)(S)

as stated in **LRFD [Table 4.6.2.1.3-1]**

Where:

S	=	Spacing of supporting components (ft)
+M	=	Positive moment
-M	=	Negative moment

36.4.6.2 Depth of Fill Greater than or Equal to 2.0 ft.

Where the depth of fill is 2.0 ft or greater, follow **LRFD [3.6.1.2.6b-c]**. The effect of multiple lanes shall be considered. Use the multiple presence factor, m, as required per **LRFD [3.6.1.1.2]**.

36.4.6.2.1 Case 1 – Traffic Travels Parallel to Span

When the traffic travels primarily parallel to the span, follow **LRFD [3.6.1.2.6b]**.

For live load distribution <u>transverse</u> to span, the wheel/axle load interaction depth, H_{int-t}, shall be:

$$H_{\text{int}-t} = \frac{S_w - W_t/12 - 0.06D/12}{LLDF} \quad \text{(ft)}$$

where $H < H_{int-t}$ (no lateral interaction); then $W_w = W_t/12 + LLDF \cdot (H) + 0.06 \cdot (D/12)$

where $H \geq H_{int-t}$ (lateral interaction); then $W_w = W_t/12 + S_w + LLDF \cdot (H) + 0.06 \cdot (D/12)$

For live load distribution <u>parallel</u> to span, the wheel/axle load interaction depth H_{int-p} shall be:

$$H_{\text{int}-p} = \frac{S_a - \ell_t/12}{LLDF} \quad \text{(ft)}$$

where $H < H_{int-p}$ (no longit. interaction); then $\ell_w = \ell_t/12 + LLDF \cdot (H)$

where $H \geq H_{int-p}$ (longit. interaction); then $\ell_w = \ell_t/12 + S_a + LLDF \cdot (H)$

Where:

D	=	Clear span of the culvert (in)
H	=	Depth of fill from top of culvert to top of pavement (in)
H_{int-t}	=	Wheel interaction depth transverse to span (ft)
H_{int-p}	=	Axle interaction depth parallel to span (ft)
LLDF	=	Live load distribution factor per **LRFD [Table 3.6.1.2.6a-1]**; (1.15)
W_t	=	Width of tire contact area, per **LRFD [3.6.1.2.5]**; (20 in)
ℓ_t	=	Length of tire contact area, per **LRFD [3.6.1.2.5]**; (10 in)
S_w	=	Wheel spacing; (6.0 ft)

S_a	=	Axle spacing (ft)
W_w	=	Live load patch width at depth H (ft)
ℓ_w	=	Live load patch length at depth H (ft)

$A_{LL} = \ell_w \cdot W_w$

Where:

A_{LL}	=	Rectangular area at depth H (ft^2)

The live load vertical crown pressure shall be:

$$P_L = \frac{P(1 + IM/100)(m)}{A_{LL}}$$

Where:

IM	=	Dynamic load allowance (%); (see 36.4.8)
m	=	Multiple presence factor per **LRFD [3.6.1.1.2]**
P	=	Live load applied at surface on all interacting wheels (kip)
P_L	=	Live load vertical crown pressure (ksf)

The longitudinal and transverse distribution widths for depths of fill greater than or equal to 2.0 feet are illustrated in Figure 36.4-4.

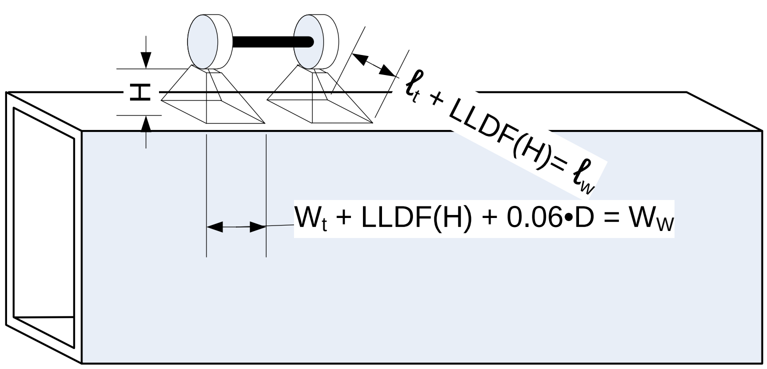


Figure 36.4-4
Distribution of Wheel Loads, Depth of Fill ≥ 2.0 feet (no lateral interaction)

36.4.6.2.2 Case 2 – Traffic Travels Perpendicular to Span

When traffic travels perpendicular to the span, live load shall be distributed to the top slab as described in **LRFD [3.6.1.2.6c]**.

36.4.7 Live Load Soil Pressures

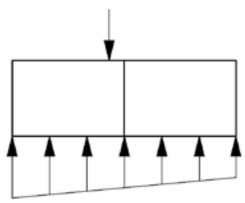


Figure 36.4-5
Vertical Soil Pressure under Culvert

The soil pressure on the bottom of the box is determined by moving the live load across the box. Find the location where the live load causes the maximum effects on the top slab of the box. At that location, determine the soil pressure diagram that will keep the system in equilibrium. Use the effects of this soil pressure in the bottom slab analysis.

36.4.8 Dynamic Load Allowance

Dynamic load allowance decreases as the depth of fill increases. **LRFD [3.6.2.2]** states that the impact on buried components shall be calculated as:

$$IM = 33(1.0 - 0.125(D_E)) \geq 0\%$$

Where:

D_E = Minimum depth of earth cover above the structure (ft)

36.4.9 Location for Maximum Moment

Create influence lines and use notional loading to determine the location for maximum moment. In this analysis, include cases for variable axle spacing and reverse axle order for unsymmetrical loading conditions.

For notional vehicles, only the portion of the loading that contributes to the effect being maximized is included. This is illustrated in Figure 36.4-6.

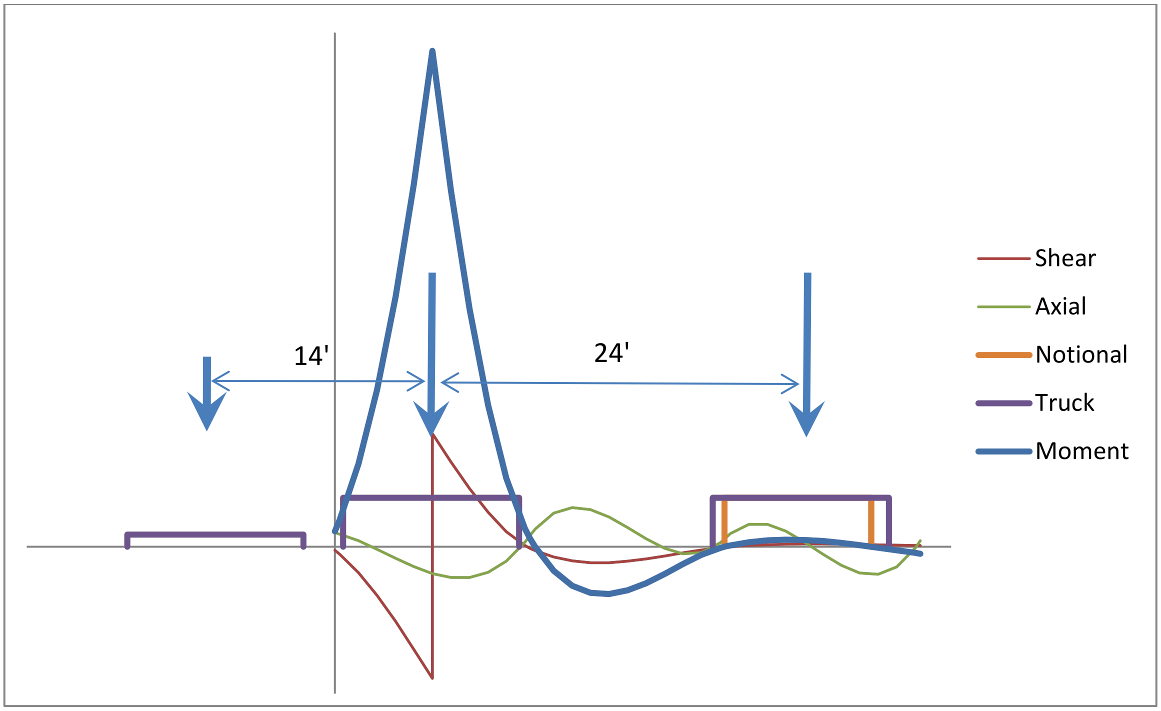


Figure 36.4-6
Application of Notional Loading using Influence Lines

The maximum positive moment results when the middle axial load is centered at the first positive peak while the variable rear axial spacing is 24 feet. Only the portion of the rear axial load in the positive region of the moment influence line is considered. The middle axial load and the portion of the rear axial in the positive region of the moment influence line are loaded on the shear and axial influence lines to compute the corresponding effects. Both positive and negative portions of the shear and axial influence lines are used when computing the corresponding effects. This process is repeated for maximizing the negative moment, shear and axial effects and computing the corresponding effects.

36.5 Design Information

Sidesway of the box is not considered because of the lateral support of the soil.

The centerline of the walls and top and bottom slabs are used for computing section properties and dimensions for analysis.

WisDOT Policy Item:

For skews 20 degrees or less, place the reinforcing steel along the skew. For skews over 20 degrees, place the reinforcing steel perpendicular to the centerline of box.

Culverts are analyzed as if the reinforcing steel is perpendicular to the centerline of box for all skew angles.

The minimum thickness of the top and bottom slab is 6½ inches. For pedestrian underpasses and slabs with fills less than 2 feet, the minimum thickness of the top slab should be 1 foot. Minimum wall thickness is based on the inside opening of the box (height) and the height of the apron wall above the floor. Use the following table to select the minimum wall thickness that meets or exceeds the three criteria in the table.

Minimum Wall Thickness (Inches)	Cell Height (Feet)	Apron Wall Height Above Floor (Feet)
8	< 6	< 6.75
9	6 to < 10	6.75 to < 10
10	10 to > 10	10 to < 11.75
11		11.75 to < 12.5
12		12.5 to 13

Table 36.5-1
Minimum Wall Thickness Criteria

All slab thicknesses are rounded to the next largest ½ inch.

Top and bottom slab thicknesses are determined by shear and moment requirements. Slab thickness shall be adequate to carry the factored shear without shear reinforcement.

All bar steel is detailed as being 2 inches clear with the following exceptions:

- The bottom steel in the bottom slab is detailed with 3 inches clear
- The top steel in the top slab of a box culvert with no fill is detailed with 2½ inches clear

A haunch is provided only when the slab depth required at the interior wall is more than 2 inches greater than that required for the remainder of the span. Only 45º haunches shall be used. Minimum haunch depth and length is 6 inches. Haunch dimensions are increased in 3 inch increments.

The slab thickness required is determined by moment or shear, whichever governs.

The shear in the top and bottom slabs is assumed to occur at a distance "d" from the face of the walls. The value for "d" equals the distance from the centroid of the reinforcing steel to the face of the concrete in compression. When a haunch is used, shear must also be checked at the end of the haunch.

For multi-cell culverts make interior and exterior walls of equal thickness.

Culverts shall be designed for the range of fill between the shoulders of the roadway. To accommodate future widening of the roadway, reduced sections may not be used on the ends of the culvert where there is less fill. Exceptions may be made with the approval of the Bureau of Structures where the culvert has high fills and a reduced section could be used for at least two panel pours per end of culvert. Culvert extensions shall be designed for the same range of fills as the original culvert. The extension design shall not have lower capacity than the original culvert. Maximum length of panel pour is 40 feet.

Barrel lengths are based on the roadway sections and wing lengths are based on a minimum 2 1/2:1 slope of fill from the top of box to apron. Consideration shall be given to match the typical roadway cross slope.

Dimensions on drawings are given to the nearest ¼ inch only.

36.6 Detailing of Reinforcing Steel

To calculate the required bar steel area and cutoff points a maximum positive and negative moment envelope is computed. It is assumed that the required bar lengths in the top slab are longer than those in the bottom slab. Therefore, cutoff points are computed for the top slab and are also used in the bottom slab.

36.6.1 Bar Cutoffs

Per **LRFD [5.10.8.1.2a]**, 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 [5.10.8.2]**) beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure.

Per **LRFD [5.10.8.1.2b]**, 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 [5.10.8.1.2c]**, 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

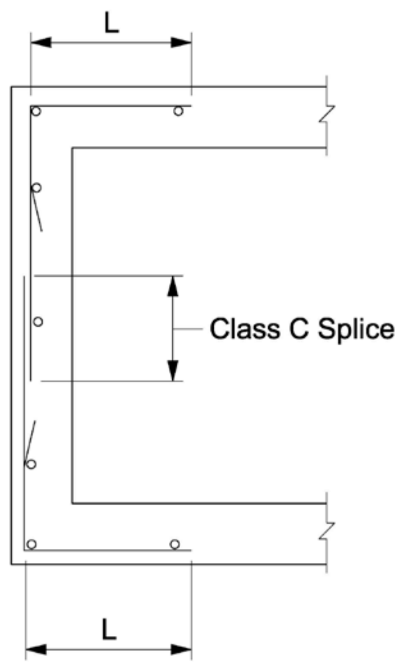


Figure 36.6-1
Layout of Corner Steel

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

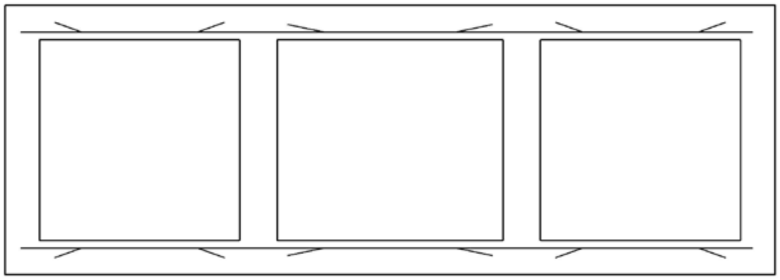


Figure 36.6-2
Layout of Positive Moment Steel

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

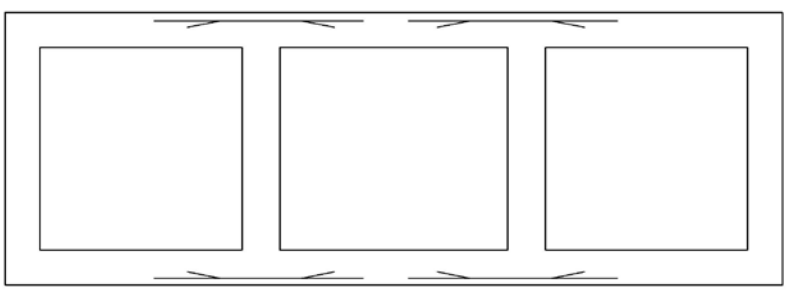


Figure 36.6-3
Layout of Negative Moment Steel

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 [12.11.5.2],** 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

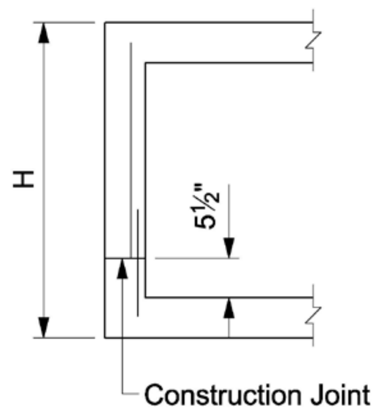


Figure 36.6-4
Layout of Exterior Wall Steel

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

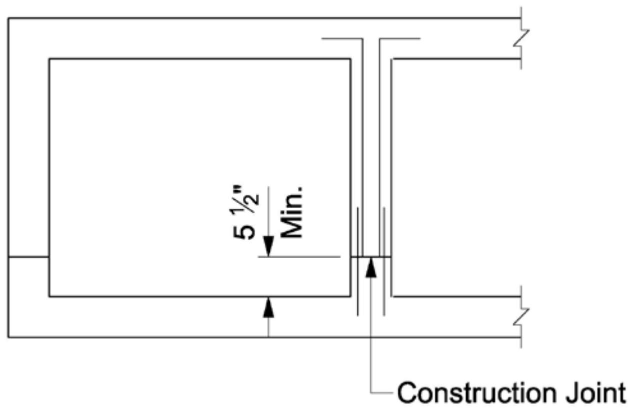


Figure 36.6-5
Layout of Interior Wall Steel

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 [5.12.2.1]**, 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]**)

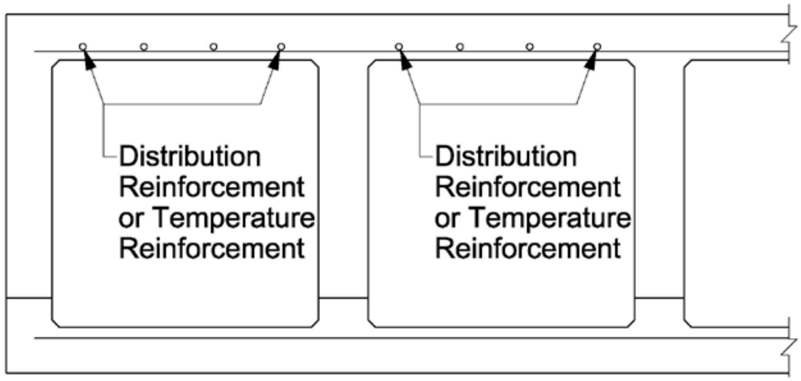


Figure 36.6-6
Layout of Distribution Steel

36.6.8 Shrinkage and Temperature Reinforcement

Shrinkage and temperature reinforcement is required on all inside culvert faces, negative moment regions in top slabs, and on both wingwall faces in each direction that does not already have strength or distribution reinforcement. Shrinkage and temperature reinforcement is not required on the outside (soil) face for culvert barrels. This includes exterior walls, the bottom of the bottom slab, and in some cases the top face of the top slab in the positive moment region. Per **LRFD [12.11.5.3.1]**, provide shrinkage and temperature reinforcement near the inside surfaces of walls and slabs in accordance with **LRFD [5.10.6]**, 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.

Shrinkage and temperature reinforcement shall use a minimum of #4 bars at 18 inch centers in both directions.

36.7 Box Culvert Aprons

Five types of box culvert aprons are used. They are referred to as Type A, B, C, D and E. The angle that the wings make with the direction of stream flow is the main difference between the five types. The allowable headwater and other hydraulic requirements are what usually determine the type of apron required. Physical characteristics at the site may also dictate a certain type. For hydraulic design of different apron types see Chapter 8.

36.7.1 Type A

Type A, because of its poor hydraulic properties, is generally not used except for cattle or pedestrian underpasses.

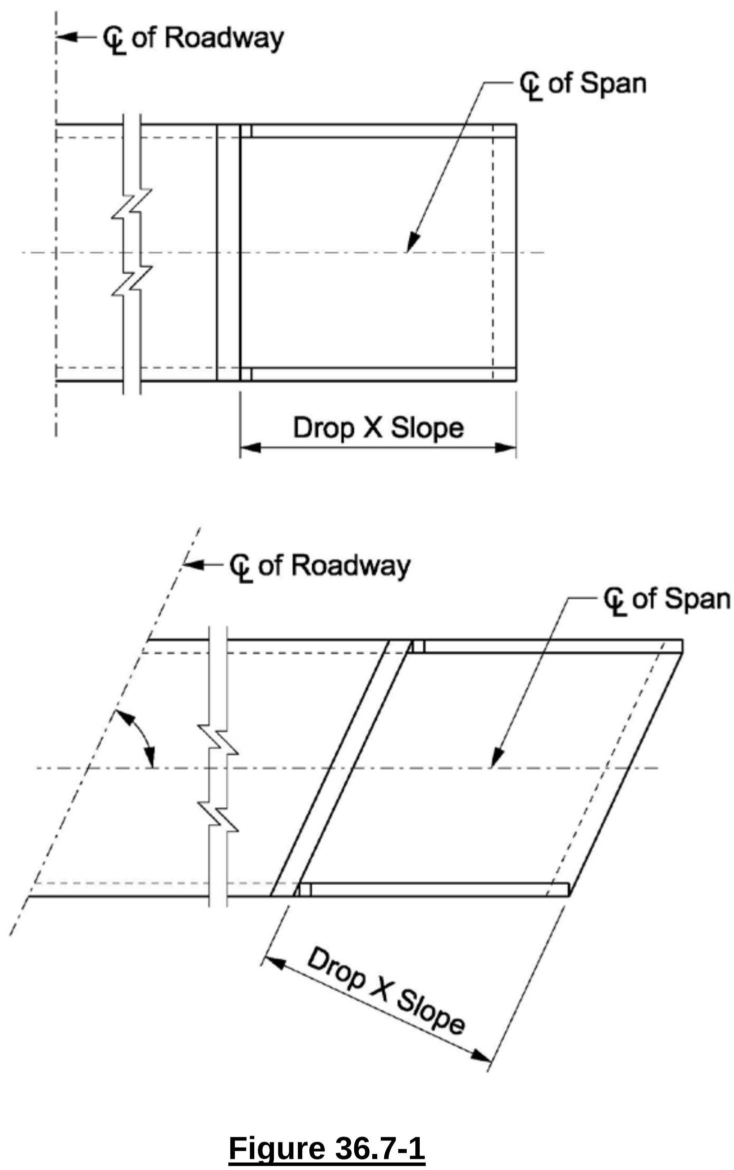
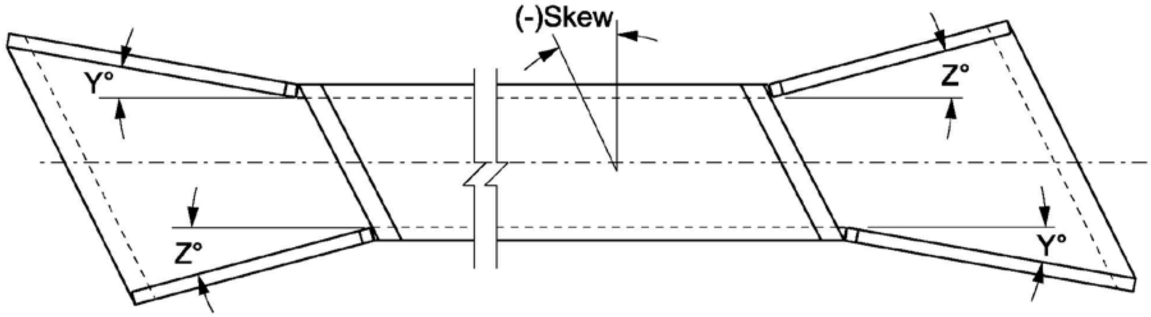


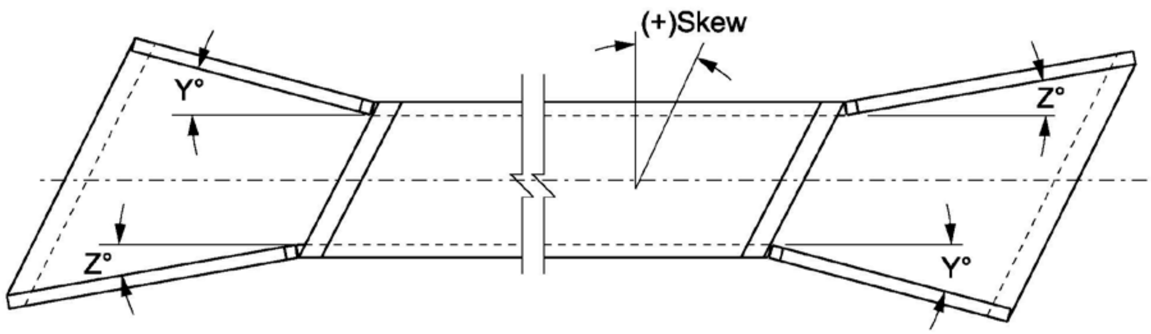
Figure 36.7-1
Plan View of Type A

36.7.2 Type B, C, D

Type B is used for outlets. Type C & D are of equal efficiency but Type C is used most frequently. Type D is used for inlets when the water is entering the culvert at a very abrupt angle. See Figure 36.7-2 for Wing Type B, C and D for guidance on wing angles for culvert skews.



Skew		Wing Type B		Wing Type C		Wing Type D	
Greater Than	To Include	Angle Y	Angle Z	Angle Y	Angle Z	Angle Y	Angle Z
0°	7.5°	15°	15°	30°	30°	45°	45°
7.5°	15.0°	15°	15°	25°	30°	40°	45°
15.0°	22.5°	10°	15°	25°	30°	35°	45°
22.5°	37.5°	10°	15°	20°	30°	30°	45°
37.5°	45.0°	10°	15°	15°	30°	25°	45°
45.0°	52.5°	5°	15°	15°	30°	20°	45°
52.5°	67.5°	5°	15°	10°	30°	15°	45°
67.5°	75.0°	5°	15°	5°	30°	10°	45°
75.0°	82.5°	0°	15°	5°	30°	5°	45°
82.5°	90.0°	0°	15°	0°	30°	0°	45°



Skew		Wing Type B		Wing Type C		Wing Type D	
Greater Than	To Include	Angle Y	Angle Z	Angle Y	Angle Z	Angle Y	Angle Z
0°	7.5°	15°	15°	30°	30°	45°	45°
7.5°	15.0°	15°	15°	30°	25°	45°	40°
15.0°	22.5°	15°	10°	30°	25°	45°	35°
22.5°	37.5°	15°	10°	30°	20°	45°	30°
37.5°	45.0°	15°	10°	30°	15°	45°	25°
45.0°	52.5°	15°	5°	30°	15°	45°	20°
52.5°	67.5°	15°	5°	30°	10°	45°	15°
67.5°	75.0°	15°	5°	30°	5°	45°	10°
75.0°	82.5°	15°	0°	30°	5°	45°	5°
82.5°	90.0°	15°	0°	30°	0°	45°	0°

Figure 36.7-2
Wing Type B, C, D (Angles vs. Skew)

36.7.3 Type E

Type E is used primarily in urban areas where a sidewalk runs over the culvert and it is necessary to have a parapet and railing along the sidewalk. For Type E the wingwalls run parallel to the roadway just like the abutment wingwalls of most bridges. It is also used where Right of Way (R/W) is a problem and the aprons would extend beyond the R/W for other types. Wingwall lengths for Type E wings are based on a minimum channel side slope of 1.5 to 1.

36.7.4 Wingwall Design

Culvert wingwalls are designed using a 1 foot surcharge height, a unit weight of backfill of 0.120 kcf and a coefficient of lateral earth pressure of 0.5, as discussed in 36.4.3. When the wingwalls are parallel to the direction of traffic and where vehicular loads are within ½ the wall height from the back face of the wall, design using a surcharge height representing vehicular load per **LRFD [Table 3.11.6.4-2]**. Load and Resistance Factor Design is used, and the load factor for lateral earth pressure of γ_{EH} = 1.69 is used, based on past design experience. The lateral earth pressure was conservatively selected to keep wingwall deflection and cracking to acceptable levels. Many wingwalls that were designed for lower horizontal pressures have experienced excessive deflections and cracking at the footing. This may expose the bar steel to the water that flows through the culvert and if the water is of a corrosive nature, corrosion of the bar steel will occur. This phenomena has led to complete failure of some wingwalls throughout the State.

For wing heights of 7 feet or less determine the area of steel required by using the maximum wall height and use the same bar size and spacing along the entire wingwall length. The minimum amount of steel used is #4 bars at 12 inch spacing. Wingwall thickness is made equal to the barrel wall thickness.

For wing heights over 7 feet the wall length is divided into two or more segments to determine the area of steel required. Use the same bar size and spacing throughout each segment, as determined by using the maximum wall height in the 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 over-consolidated, reference is made to **LRFD [10.6.2.4.3]** to estimate consolidation settlement.

Further description for the above equations and consolidation test can be found in most textbooks on soil mechanics.

For preliminary investigations C_c can be determined from the following approximate formula, found in most soil mechanics textbooks:

Non organic soils: $C_c = 0.007\,(LL-10)$

Where:

LL	=	Liquid limit expressed as whole number.

If the in-place moisture content approaches the plastic limit the computed C_c is decreased by 75%. If the in-place moisture content is near the liquid limit use the computed value. If the in-place moisture content is twice the liquid limit the computed C_c is increased by 75%. For intermediate moisture contents the percent change to the computed C_c is determined from a straight line interpolation between the corrections mentioned above.

If settlements computed by using the approximate value of C_c exceed 1.5 feet, a consolidation test is performed. As in-place moisture content approaches twice the liquid limit, settlement is caused by a local shear failure and the consolidation equation is no longer applicable.

The consolidation equation is applied to only compressible silts and clays. Sands are of a lower compressibility and no culvert camber is required until the fill exceeds 25 feet. When the fill exceeds 25 feet for sand, a camber of 0.01 feet per foot of fill is used.

36.8.2 Configuration of Camber

The following guides are to be followed when detailing camber.

- It is unnecessary to provide gradual camber. "Brokenback" camber is closer to the actual settlement which occurs.
- Settlement is almost constant from shoulder point to shoulder point. It then reduces to the ends of the culvert at the edge of the fill.
- The ends of the culvert tend to come up if side slopes are steeper than 2½ to 1. With 2 to 1 side slopes camber is increased 10% to compensate for this rise.

36.8.3 Numerical Example of Settlement Computation

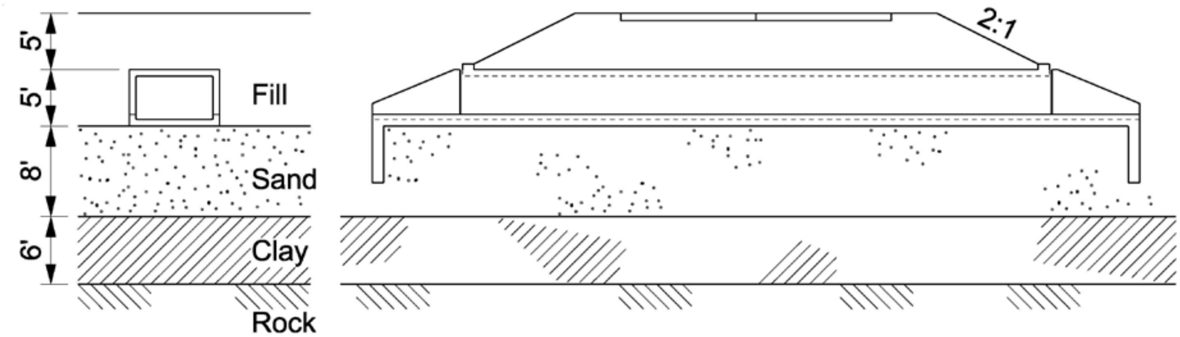


Figure 36.8-1
Soil Strata under Culvert

A box culvert rests on original ground consisting of 8 feet of sand and 6 feet of clay over bedrock. Estimate the settlement of the culvert if 10 feet of fill is placed on the original ground after the culvert is constructed. The in-place moisture content and liquid limit equal 40%. The initial void ratio equals 0.98. The unit weight of the clay is 105 pcf and that of the fill and sand is 110 pcf. There is no water table.

σ'_o = (8 ft)(110 pcf) + (3 ft)(105 pcf) = 1195 psf

$\sigma'_f = \sigma'_o$ + (10 ft)(110 pcf) = 1195 psf + 1100 psf = 2295 psf

C_c = 0.007 (40-10) = 0.21 (approximate value)

$$S_c = \left[\frac{H_c}{1+e_o}\right] c_c \log_{10}\left[\frac{\sigma'_f}{\sigma'_o}\right] = \frac{6\text{ft}}{1+0.98} 0.21 * \log_{10}\left[\frac{2295\text{psf}}{1195\text{psf}}\right] = 0.18\text{ft}$$

36.9 Box Culvert Structural Excavation and Structure Backfill

All excavations for culverts and aprons, unless on bedrock or fill, are to include a 6 inch minimum undercut and backfilled with structural backfill, as described in the specification. This undercut is for construction purposes and provides a solid base for placing reinforcement and pouring the bottom slab. For fill sections, it is assumed that placed fills provide a solid base and structural backfill is not needed. For cut sections, deeper under cuts may be warranted to mitigate differential settlement.

All volume excavated and not occupied by the new structure should be backfilled with structure backfill for the full length of the box culvert, including the apron.

See Standard Detail 9.01 – Structure Backfill Limits and Notes – for typical pay limits and plan notes.

36.10 Box Culvert Headers

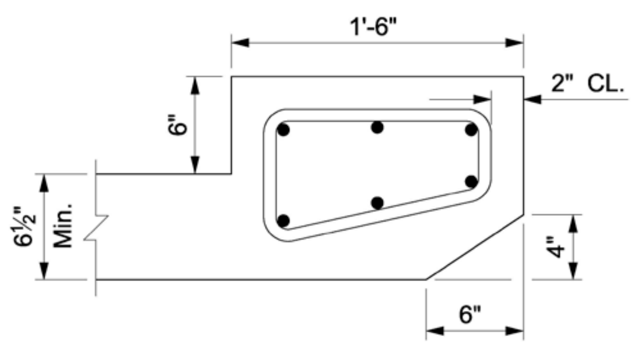
For skews of 20 degrees and less the main reinforcing steel is parallel to the end of the barrel. A header is not required for structural purposes but is used to prevent the fill material from spilling into the apron. A 12 inch wide by 6 inch high (above the top of top slab) header with nominal steel is therefore used for skews of 20 degrees and less on the top slab. No header is used on the bottom slab.

For skews over 20 degrees the main reinforcing is not parallel to the end of the barrel. The positive reinforcing steel terminates in the header and thus the header must support, in addition to its own dead load, an additional load from the dead load of the slab and fill above it. A portion of the live load may also have to be supported by the header.

The calculation of the actual load that a header must support becomes a highly indeterminate problem. For this reason a rational approach is used to determine the amount of reinforcement required in the headers. The design moment capacity of the header must be equal to or greater than 1.25 times the header dead load moment (based on simple span) plus 1.75 times a live load moment from a 16 kip load assuming 0.5 fixity at ends.

To prevent a traffic hazard, culvert headers are designed not to protrude above the ground line. For this reason the height of the header above the top of the top slab is typically selected to be 6 inches. The width of the header is standardized at 18 inches.

The header in the following figure gives the design moment capacities listed using d = 8.5 inches.



Bar Size	Moment Capacity ft.-kip
#6	46.1
#7	60.7
#8	76.5
#9	92.1
#10	109.1

Figure 36.10-1
Header Details (Skews > 20°)

The following size bars are recommended for the listed header lengths where "Header Length" equals the distance between C/L of walls in one cell measured along the skew.

Header Length	Bar Size [1]
To 11'	#7
Over 11' to 14'	#8
Over 14' to 17'	#9
Over 17' to 20'	#10

Table 36.10-1
Header Reinforcement

[1] Use the bar size listed in each header and place 3 bars on the top and 3 bars on the bottom. Use a header on both the top and bottom slab. See the Standard *Box Culvert Details* in Chapter 36.

Where headers greater than 6 inches in height are used to retain roadway fill, the top slab shall be designed to handle the bending moment transmitted from the header. Additional reinforcement may be required.

Where barriers are placed on top of the culvert header, the barrier, header, and top slab shall be designed for vehicular impact forces.

36.11 Plan Detailing Issues

36.11.1 Weep Holes

Investigate the need for weep holes for culverts in cohesive soils. These holes are to relieve the hydrostatic pressure on the sides of the culverts. Where used, place the weep holes 1 foot above normal water elevation but a minimum of 1 foot above the lower sidewall construction joint. Do not place weep holes closer than 1 foot from the bottom of the top slab.

36.11.2 Cutoff Walls

Where dewatering the cutoff wall in sandy terrain is a problem, the concrete may be poured in the water. Place a note on the plans allowing concrete for the cutoff wall to be placed in the water.

36.11.3 Nameplate

Designate a location on the wingwall for placement of the nameplate. Locate nameplate on the first right wing traveling in the Cardinal direction (North/East).

36.11.4 Plans Policy

If a cast-in-place reinforced concrete box culvert is used, full plans must be provided and sealed by a professional engineer to the Bureau of Structures for approval. The plans must be in accordance with the *Bridge Manual* and Standards.

36.11.5 Rubberized Membrane Waterproofing

When required by the Standard Details, place the bid item "Rubberized Membrane Waterproofing" on the final plans. The quantity is given square yards.

36.12 Precast Four-Sided Box Culverts

In general, structural contractors prefer cast-in-place culverts while grading contractors prefer precast culverts. Precast culverts have been more expensive than cast-in-place culverts in the past, but allow for reduced construction time. Box culverts that are less than 20 square feet 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 vice 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 Example

Table of Contents

38.1 Introduction

The principles of designing railroad structures are similar to those for structures carrying highways. However, structures carrying railways have much heavier loadings than those subject to highway loadings due to increased dead load, live load and impact required for railways.

The general features of design, loadings, allowable stresses, etc., for railway structures are controlled by the specifications of the American Railway Engineering and Maintenance-of-Way Association (AREMA). The different railroad companies vary somewhat in their interpretation and application of these specifications as stated in the *AREMA Manual for Railway Engineering* (hereafter referred to as *AREMA Manual).* Requirements for railroad structures vary with the railroad company whose tracks are carried by the structure, and are sometimes varied by the same company in different locations. The *AREMA Manual* provides for design of railroad structures using Allowable Stress Design (ASD) and Load Factor Design (LFD) methods. The Load and Resistance Factor Design (LRFD) method is currently not used. Designers should bear in mind that specifications were developed for more or less typical conditions. If a structure is unusual in some respects, designers should use their best engineering judgment in selection of proper design criteria. Most railroad companies permit and prefer high strength bolted or shop welded steel plate girders, reinforced concrete or prestressed concrete members in bridge construction.

Safety of the traveler on the highway under the structure and uniformity of track surface dictates that the full ballast section of the railway be carried on the structure. The relatively heavy loadings and high impact factor together with the span and clearance requirements usually found in underpass structures, practically limit the choice of materials for the superstructure to structural steel. The floor under the ballast may be steel plate or reinforced concrete and the substructures could be reinforced concrete or structural steel as conditions warrant.

The *AREMA Manual* covers all phases of railway design, construction, maintenance and operation. It is divided into sections and chapters. Chapter 8, Concrete Structures and Foundations (Volume 2), governs the design and construction of plain and reinforced concrete members, rigid concrete structures, retaining walls, pile foundations, substructures of railway structures, etc. Chapter 15 - Steel Structures (Volume 2), governs the design and construction of steel railroad structures.

In this chapter, reference will be made to specific articles of the *AREMA Manual* as required.

The AREMA specifications are revised annually and it is essential that the latest revisions be used. The *AREMA Manual* is a guideline only and should be followed as a starting point in design.

Railroad companies are essentially conservative as their primary interest is the safety of their trains and human lives. Their requirements are usually based upon their operating experience and are set up with that interest in view.

38.2 Design Specifications and Design Aids

38.2.1 Specifications

Allowable stresses are provided in different chapters and sections of the *AREMA Manual*.

Refer to the design, construction, maintenance and operation related materials as presented in the stated sections of the following specifications:

American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering

- Chapter 8 - Concrete Structures and Foundations (Volume 2)
- Chapter 15 - Steel Structures – (Design, Fabrication and Construction) (Volume 2)
- Chapter 28 – Clearances - (Horizontal and Vertical) (Volume 4)

AASHTO Standard Specifications for Highway Bridges, 17th Edition

Wisconsin Standard Specifications for Highway and Structure Construction

38.2.2 Design Aids

In the design of railroad structures the only short cut available is a method of computing Live Load Moments, Shears and Reactions by the use of tables which can be found in Section 1.15 of Chapter 15, part 1 of the *AREMA Manual*. This table reflects Cooper E80 Live Loading shown in Figure 38.3-5. All the moment, shear and reaction values are for one rail (one-half track load) only and all the values can be prorated (directly proportional) for smaller or larger Cooper's E live loadings.

Floor beam spacings in through plate-girder railroad structures may be determined by a number of things, but consideration should be given to the transverse stiffener spacings of the girders. It is very convenient to have the floor beam spacing in multiples of stiffener spacings.

For ballasted structures, all lateral forces will be carried by the steel ballast plate which is extremely rigid and lateral bracing will not be required.

38.2.3 Horizontally Curved Structures

The latest AREMA specifications as well as individual railroad company's interpretation and application of the *AREMA Manual* should be followed in designing and detailing curved structures. There is considerable information available on designing curved steel girders. Most of the methods require computer programs that may be difficult to use. The Approximate Method of Design developed by USS Corporation is an accepted approach for horizontally curved girders.

38.2.4 Railroad Approval of Plans

There is a need to get the individual railroad company's unique design requirements. Smaller companies such as Wisconsin & Southern may rely on AREMA requirements and DOT experience.

Prior to starting the preliminary design, the Bureau of Structures (BOS) should receive the railroad company's current standards and design policy guidelines.

Before the preliminary plan is prepared, the Regional Project Manager, BOS and Bureau of Rails and Harbors (BRH) should review the particular railroad company's design standards for compliance with 23 CFR (Code of Federal Regulations) and DOT policy, and for compatibility and practicality with unique project features.

The preliminary structure plan should be prepared and submitted to the railroad company for approval after the above steps have been completed.

Detailed structure design should not begin until the railroad company has approved the preliminary plan.

The bridge designer should work directly with the railroad's bridge engineering office where interpretation of requirements or clarification of design details is needed.

The final structure plan and special provisions need to be sent to and approved by the railroad company before the project is authorized for letting.

38.3 Design Considerations

38.3.1 Superstructure

38.3.1.1 Methods of Design, Selection Type and Superstructure General

The preferred types of railroad structures are as follows:

- Rolled or welded girders for spans of 50 feet or less
- Bolted or welded plate girders for spans over 50 to 150 feet
- Bolted or welded trusses for spans over 150 feet

The superstructures of grade separations carrying railroad traffic are usually of beam and girder construction. The spans are generally too short for economical use of trusses and other factors, such as appearance, maintenance, etc., discourage their use.

Floor systems in beam and girder construction, for moderate spans, may be divided into two general classes:

- One-way Floor System
- Two-way Floor System

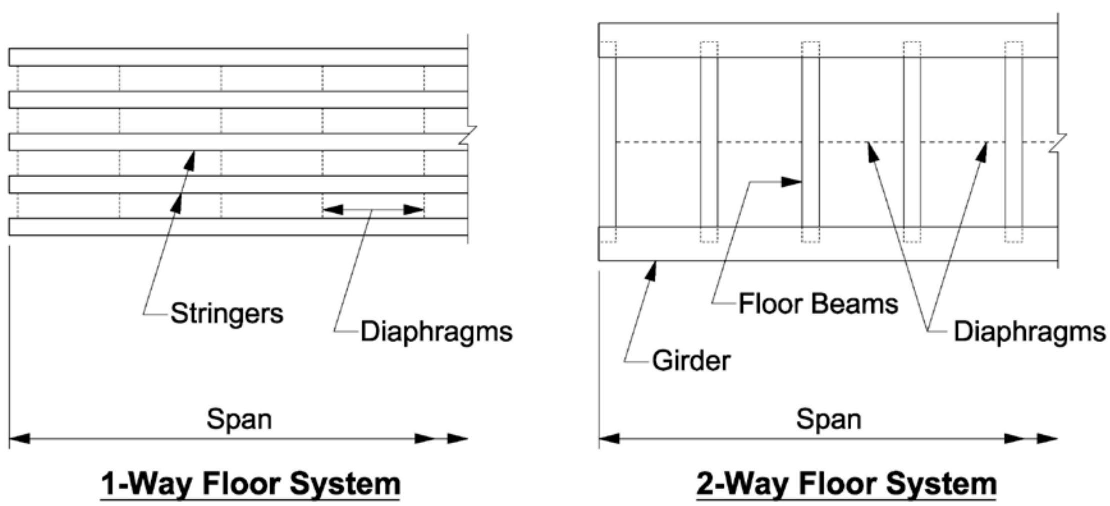


Figure 38.3-1
Types of Floor Systems

The One-way floor system is always a deck structure and is particularly adaptable for structures carrying several tracks or subject to future widening or other controls which make a

The type and location of the various constructional details are categorized in Table 15-1-9 and illustrated in Figure 15-1-5 *AREMA Manual*. The stress range for other than Fracture Critical Members shall not exceed the allowable fatigue stress range, $S_R$fat, listed in Table 15-1-10.

The stress range for Fracture Critical Members shall not exceed the allowable fatigue stress range $S_R$fat, listed in Table 15-1-10 (see Note 2) *AREMA Manual*.

38.3.1.14 Live Load Deflection

The deflection of the structure shall be computed for the live loading plus impact loading condition producing the maximum bending moment at mid-span for simple spans. In this computation, gross moment of inertia shall be used for flexural members and gross area of members for trusses. For members with perforated cover plates, the effective area shall be used.

The structure shall be so designed that the computed deflection shall not exceed 1/640 of the span length, center to center of bearings for simple spans.

38.3.1.15 Loading Combinations on Railroad Structures

Every component of superstructure and substructure should be proportioned to resist all combinations of forces applicable to the type of structure and its site. Members subject to stresses resulting from dead load, live load, impact load and centrifugal force shall be designed so that the maximum stresses do not exceed the basic allowable stresses of Section 1.4, and the stress range does not exceed the allowable fatigue stress range allowed by AREMA specifications.

The basic allowable stresses of Section 1.4 shall be used in the proportioning of members subject to stresses resulting from wind loads only, as specified in *AREMA Manual*, Article 1.3.8.

With the exception of floorbeam hangers, members subject to stresses from other lateral or longitudinal forces, as well as to the dead load, live load, impact and centrifugal forces may be proportioned for 125% of the basic allowable unit stresses, without regard for fatigue. But the section should not be smaller than required with basic unit stresses or allowable fatigue stresses when those lateral or longitudinal forces are not present.

Increase in allowable stress permitted by the previous paragraph shall not be applied to allowable stress in high strength bolts.

38.3.1.16 Basic Allowable Stresses for Structures

Design of steel railroad structures usually is based on a working stress level that is some fraction of the minimum yield strength of the material. This value commonly is 0.55, allowing a safety factor of 1.82 against yield of the steel. The basic allowable stresses for structural steel, rivets, bolts and pins to be used in proportioning the parts of a structure are furnished in Table 15-1-12 in the *AREMA Manual* Chapter 15.

38.3.1.17 Length of Cover Plates and Moment Diagram

The dead load moment diagram is a parabola with mid-ordinate showing the maximum dead load moment. Determination of the exact shape of the envelope for the live load moment involves long and tedious calculations. The procedure consists of dividing the span into parts and finding the maximum moment at each section. The smaller the divisions, the more accurate the shape of the curve and the more involved and tedious the calculations.

Fortunately, a parabola with mid-ordinate equal to the tabular value for maximum moment, Section 1.15, *AREMA Manual* Chapter 15, very nearly encloses the envelope. Therefore the shape of the moment diagram of DL + LL + I is parabolic for all practical purposes. Knowing the maximum ordinate, the designer can compute the other values and draw the moment curve.

The resisting moment diagram can be superimposed upon actual moment diagram described above. The theoretical end of cover plates can be determined from these moment envelopes.

The AREMA specifications require that flange plates shall extend far enough to develop the capacity of the plate beyond the theoretical end. This method of determining the theoretical end of cover plates, on girders proportioned for deflection is not exact, but is acceptable for design purposes.

38.3.1.18 Charpy V-Notch Impact Requirements

Recognizing the need to prevent brittle fracture failures of main load carrying structural components, AREMA specifications have provisions for Charpy V-Notch impact testing and the values for steel other than fracture critical members are tabulated in Table 15-1-2 in *AREMA Manual*.

The design requirements for materials of Fracture Critical Members shall further comply with the Fracture Control Plan specified in *AREMA Manual* Chapter 15, Section 1.14. The Engineer shall designate on the plans which members or member components fall in the category of Fracture Critical Members.

38.3.1.19 Fracture Control Plan for Fracture Critical Members

For purposes of the Fracture Control Plan, Fracture Critical Members or member components (FCM's) are defined as those tension members or tension components of members whose failure would be expected to result in collapse of the structure or inability of the structure to perform its design function.

AREMA specifications have elaborate descriptions of the Fracture Control Plan which has special requirements for the materials, fabrication, welding, inspection and testing of Fracture Critical Members and member components in steel railway structures. The provisions of this plan are to:

- Assign responsibility for designating which steel railway structure members or member components, if any, fall in the category of "Fracture Critical".

- Require that fabrication of FCM or member components be done in plants having personnel, organization, experience, procedures, knowledge and equipment capable of producing quality workmanship.

- Require that all welding inspectors demonstrate their competency to assure that welds in FCM or member components are in compliance with this plan.

- Require that all non-destructive testing personnel demonstrate their competency to assure that tested elements of FCM or member components are in compliance with this plan.

- Specify material toughness values for FCM or member components.

- Supplement recommendations for welding contained elsewhere in AREMA specifications.

Charpy V-Notch (CVN) impact test requirements for steels in FCM's shall be always followed as given in *AREMA Manual* Table 15-1-15. Impact tests shall be in accordance with the CVN tests as governed by ASTM Designation A673 for frequency of testing P (impact). Impact tests shall be required on a set of specimens taken from each end of each plate. Wisconsin currently specifies its steel to Zone 3 when impacts are required on railroad structures. Since Wisconsin Standard Specifications say Zone 2, Zone 3 must be stated on the plans.

38.3.1.20 Waterproofing Railroad Structures

AREMA specifications on waterproofing railroad structures apply to materials and construction methods for an impervious membrane and auxiliary components to protect structures from harmful effects of water. Railroad structures which require waterproofing shall be designed so that they can be waterproofed by the methods and with the materials specified in AREMA specifications. The materials for waterproofing and the methods of application should be such as to insure that the waterproofing will be retained by bond, anchorage or other adequate means, in its original position as applied to the surface to be waterproofed.

The membrane shall consist of one of the following types, as described below.

- Minimum 3/32 inch thick butyl rubber sheeting secured with an approved adhesive.

- Heavy Duty Bituthene or Protecto Wrap M400 may be used.

- Rubberized asphalt with plastic film or 4 feet x 8 feet sheets of preformed board membrane with maximum thickness of ½ inch.

The butyl rubber sheeting, rubber membrane splicing cement and the butyl gum splicing tape shall be in accordance with the requirements for membrane waterproofing as specified in part 29 of Chapter 8 of the *AREMA Manual*. Cement for splicing rubber membrane shall be a self-vulcanizing butyl rubber compound and shall be applied at a minimum rate of 2 gallons/100 square feet.

38.3.2 Substructure

38.3.2.1 Abutments and Retaining Walls

The abutments for railroad structures are essentially bearing walls subject to lateral pressure. The design procedure is similar to that required for a retaining wall. The typical section is shown in Figure 38.3-6.

AREMA Manual Chapter 8, Part 5, governs the requirements for retaining walls. They are essentially the same as AASHTO requirements providing the backfill is of sandy material.

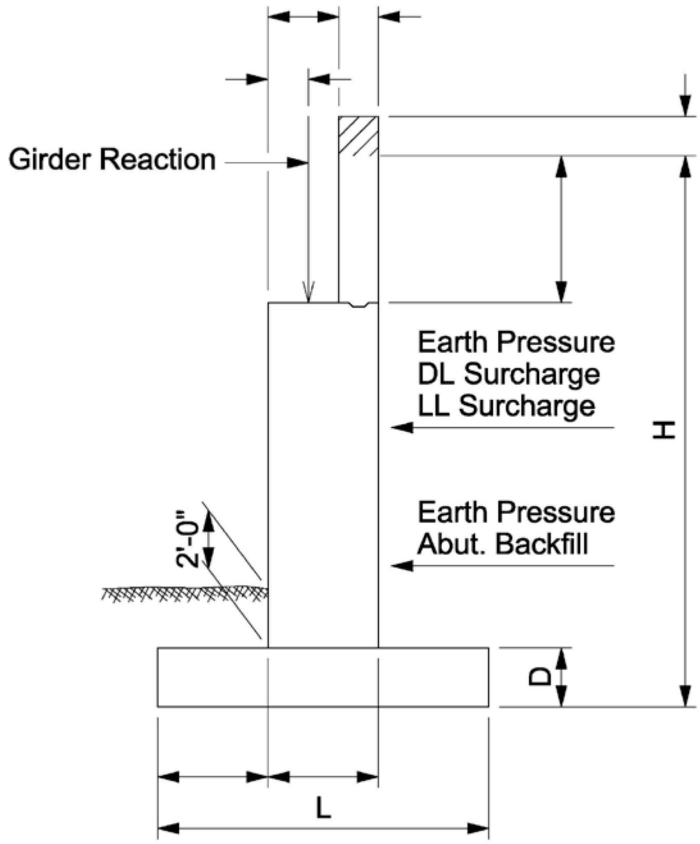


Figure 38.3-6
Typical Abutment

1. Field Survey

   Sufficient information shall be furnished, in the form of a profile and cross sections or a topographic map, to determine the structural requirements. Present grades and alignments of tracks and roads shall be indicated, together with the records of high water, low water and depth of scour, the location of underground utilities, and

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40.1 General

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

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

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

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

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

40.2 History

40.2.1 Concrete

Prior to 1975, all concrete structures were designed by the Service Load Method. The allowable design stress was 1400 psi based on an ultimate strength of 3500 psi except for deck slabs. In 1965, the allowable stress for deck slabs on stringers was reduced from 1400 to 1200 psi. The reason for the change was to obtain thicker concrete decks by using lower strength. The thicker deck was assumed to have more resistance to cracking and greater durability. During this time no changes were made in the physical properties of the concrete or the mixing quantities until 1974 when more cement was added to the mix for concrete used in bridge decks.

In 1980, the use of set retarding admixtures was required when the atmospheric temperature at the time of placing the concrete is 70°F or above; or when the atmospheric temperature is 50°F or above and it is expected that the time to place the concrete of any span or pour will require four or more hours. Retarding admixtures reduce concrete shrinkage cracking and surface spalling. Also, during the early 1980's a water reducing admixture was provided for Grades A and E concrete mixes to facilitate workability and placement.

40.2.2 Steel

Prior to 1975, Grade 40 bar steel was used in the design of reinforced concrete structures. This was used even though Grade 60 bars may have been furnished on the job. Allowable design stress was 20 ksi using the Service Load Method and 40 ksi using the Load Factor Method.

40.2.3 General

In 1978, Wisconsin Department of Transportation discovered major cracking on a two-girder, fracture critical structure, just four years after it was constructed. In 1980, on the same structure, major cracking was discovered in the tie girder flange of the tied arch span.

This is one example of the type of failures that transportation departments discovered on welded structures in the 1970's and '80's. The failures from welded details and pinned connections led to much stricter standards for present day designs.

All areas were affected: Design with identification of fatigue prone details and classifications of fatigue categories (AASHTO); Material requirements with emphasis on toughness and weldability, increased welding and fabrication standards with licensure of fabrication shops to minimum quality standards including personnel; and an increased effort on inspection of existing bridges, where critical details were overlooked or missed in the past.

Wisconsin Department of Transportation started an in-depth inspection program in 1985 and made it a full time program in 1987. This program included extensive non-destructive testing. Ultrasonic inspection has played a major role in this type of inspection. All fracture critical structures, pin and hanger systems, and pinned connections are inspected on a five-year cycle now.

40.2.4 Funding Eligibility and Asset Management

Nationally, MAP-21 (2012) and the FAST Act (2015) have moved structures asset management to a more data-driven approach. Funding restrictions with regards to Sufficiency Rating, Structural Deficiency, and Functional Obsolescence have been removed or significantly revised. In place of these past restrictions, MAP-21 requires the development and approval of a statewide Transportation Asset Management Plan (TAMP). A key part of the WisDOT TAMP is the Wisconsin Structures Asset Management System (WiSAMS).

WiSAMS is being developed as a planning tool, which analyzes current structure inspection data, projects future deteriorated structure condition, and applies the Bridge Preservation Policy Guide (BPPG) to recommend appropriate structure work actions at the optimal time. WiSAMS is a tool for regional and statewide programming, and is not designed as an in-depth scoping tool. WiSAMS may provide an estimate of the appropriate work action, but an in-depth evaluation of the actual structure condition and appropriate scope of work (SSR) and consideration of other non-structural project factors (e.g. cost and functionality) is still required.

In Wisconsin, the Local Bridge Program, through State Statute 84.18 and Administrative Rule Trans 213, is still tied to historic FHWA classifications of Sufficiency Rating, Structural Deficiency, and Functional Obsolescence.

40.3 Bridge Replacements

Bridge preservation and rehabilitation is preferred over bridge replacement if the final structure provides adequate serviceability. Ideal bridge preservation strategy is explained in the WisDOT Bridge Preservation Policy Guide (BPPG). This guide should be followed as closely as possible, considering estimated project costs and funding constraints.

See FDM 11-40-1.5 for policies regarding necessary bridge width* and structural capacity.

* If lane widening is planned as part of the 3R project, the usable bridge width should be compared to the planned width of the approaches after they are widened.

40.4 Rehabilitation Considerations

As a structure ages, rehabilitation is a necessary part of ensuring some level of acceptable serviceability; however, structure preservation as explained in the Bridge Preservation Policy Guide (BPPG) should be followed as closely as possible, considering estimated project costs and funding constraints.

The first consideration for any bridge rehabilitation decision is whether its geometry and load carrying capacity are adequate to safely carry present and projected traffic. Information which is helpful in determining structure adequacy includes structure inspection history, inventory data, traffic projections, maintenance history, capacity and route designations. The methods of rehabilitation are based on the type of structures, existing condition or rating information, the preliminary details of rehabilitation, and traffic control costs. These are important factors in considering rehabilitation options such as either a deck protection system or a deck replacement.

WisDOT policy item:

Rate the bridge using LFR provided it was designed using ASD or LFD. There are instances, however, where the LRFR rating of an existing bridge is beneficial (e.g. There is no M/M_u reduction to shear capacity using LRFR, which can affect longer-span steel bridges). Please contact the Bureau of Structures Development Section if using LRFR to rate bridges designed using ASD or LFD. Bridges designed using LRFD must be rated using LRFR.

The Regions are to evaluate bridge deficiencies when the bridge is placed in the program to ensure that rehabilitation will remove all structural deficiencies. Bureau of Structures (BOS) concurrence with all proposed bridge rehabilitation is required. See FDM 11-40-1.5 for policies regarding bridge rehabilitation.

Deck removal on prestressed girders is a concern as the contractors tend to use jackhammers to remove the deck. Contractors have damaged the top girder flanges using this process either by using too large a hammer or carelessness. With the wide-flange sections this concern is amplified. It is therefore suggested that the contractor saw cut the slab to be removed longitudinally close to the shear connectors. With the previously applied bond breaker, the slab should break free and then the contractor can clear the concrete around the shear connectors. Saw cutting needs to be closely monitored as contractors have sawn through steel girder flanges by not watching their blade depth.

In the rehabilitation or widening of bridge decks, it is often necessary to place concrete while traffic uses adjacent lanes. There has been considerable concern over the effects of traffic-induced vibrations on the quality of the concrete and its bond to existing concrete and its embedded reinforcing steel. Wisconsin bridge construction experience indicates that there are many cases where problems have occurred during deck pours with adjacent traffic. Consideration should be given to prohibiting traffic during the deck pour until concrete has taken its initial set.

The most effective way to reduce the amplitude of traffic-induced vibrations is to maintain a smooth structure approach and riding surface. Wisconsin has experienced some cracking in the concrete overlays and parapets during rehabilitation construction. The apparent cause of cracking is related to traffic impact due to rough structure approaches. Particular attention should be given to providing a smooth transition on temporary approaches and over expansion joints on all bridge decks during rehabilitation. Other causes of cracking are related to shrinkage, temperature, wind induced vibrations, concrete quality and improper curing.

Options for deck rehabilitation are as follows:

1. Asphalt Patch
2. Asphalt or Polymer Modified Asphaltic Overlay
3. Concrete or Modified Concrete Patch
4. Waterproof Membrane with an Asphalt Overlay (currently not used)
5. Concrete Overlay - Grade E Low Slump Concrete, or Micro-Silica Modified Concrete

Consider the following criteria for rehabilitation of highway bridges:

1. Interstate Bridges as Stand Alone Project
    a. Deck condition equal 4 or 5 and;
    b. Wear course or wear surface less than or equal to 3.
    c. No roadway work scheduled for at least 3 years.
2. Interstate Bridge with Roadway Work
    a. No previous work in last 10 years or;
    b. Deck Condition less than or equal 4.
    c. Wear course or wear surface less than or equal to 4.
3. Rehab not needed on Interstate Bridges if:
    a. Deck rehab work less than 10 years old.
    b. Deck condition greater than 4.
    c. Wear surface or wear course greater than or equal 4.
4. All Bridges

WisDOT policy item:

On major rehab work, build to current standards such as safety parapets, full shoulder widths, etc. Use the current Bridge Manual standards and tables. Exceptions to this policy require approval from the Bureau of Structures Development Section.

a. Evaluate cost of repeated maintenance, traffic control as well as bridge work when determining life-cycle costs.

b. Place overlays on all concrete superstructure bridges if eligible.

c. For all deck replacement work the railing shall be built to current standards.

5. All Bridges with Roadway Work

   Coordinate with the Region the required staging of bridge related work.

A number of specific guidelines are defined in subsequent sections. As with any engineering project, the engineer is allowed to use discretion in determining the applicability of these guidelines.

40.5 Deck Overlays

As a bridge deck ages, preservation and rehabilitation techniques are necessary to maximize the life of the deck and ensure a level of acceptable serviceability. Overlays can be a useful tool to extend the service life of structures. This section discusses several overlay methods, considerations, and guidelines for deck overlays. The provided information is intended for deck-girder structures and may be applicable for slab structures. Slab structures may have different condition triggers and may warrant additional considerations.

The following criteria should be met when determining if an overlay should be used:

- The structure is capable of carrying the overlay dead load
- The deck and superstructure are structurally sound
- The desired service life can be achieved with the considered overlay and existing structure
- The selected option is cost effective based on the anticipated structure life and funding constraints

Decks deteriorate at different rates depending on many factors, including deck materials, material quality, construction quality, structure geometry, exposure to deicing agents, and traffic demands. Additionally, there is a wide variance in the amount of structure preservation techniques utilized by different regions. While the deck age can be a useful parameter, it should not be the primary consideration for determining the eligibility of overlays. Recommended preservation techniques should rely heavily on quality inspection data to determine the appropriate course of action. For more information related to preservation techniques and practices, refer to the Bridge Preservation Policy Guide.

Overlays can be an effective tool to maximize the life of the deck. Figure 40.5-1 illustrates a possible preservation scenario using deck deterioration curves showing approximate deck NBI ratings at which the overlays would occur, and the benefit of performing these overlays. This scenario assumes that the underside of deck deterioration is significantly reduced due to the preservation techniques performed on the top side of the deck.

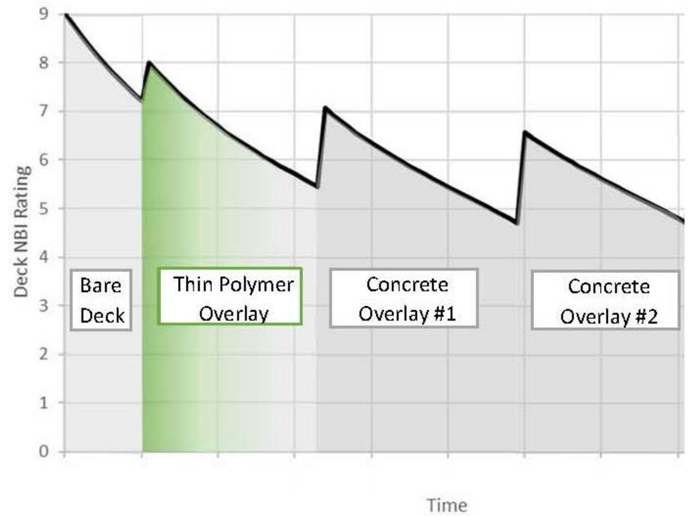

Figure 40.5-1
Deck Deterioration Curve

40.5.1 Overlay Methods

There are several commonly used overlay methods for the preservation and rehabilitation of decks. Generally, thin polymers overlays are recommended as preventative maintenance for decks with a minimal amount of deck distress. Ideally, thin polymer overlays are applied within the first couple of years to limit chloride infiltration. For decks with distress, the existing deck is typically milled and repaired with a low-slump concrete overlay as part of a more extensive bridge rehabilitation effort. For decks nearing replacement, asphaltic overlays maybe a cost effective option to improve ride quality. Refer to the following sections and Table 40.5-1 and Table 40.5-2 for a list of common overlay methods and additional information.

40.5.1.1 Thin Polymer Overlay

A thin polymer overlay (TPO) is expected to extend the service life of a bridge deck for 7 to 15 years. This overlay adds minimal dead load to the existing structure while providing an impermeable surface to prevent chlorides from infiltrating the deck. It can also be used to improve or restore friction on bridge decks.

In general, thin polymer overlays are defined as 1-inch thick or less overlays consisting of a polymer binder with aggregates and can be placed either as a multi-layer, slurry, or premixed system. Typical polymer binders are either epoxy, polyester, or methacrylate based. For WisDOT applications, TPO's consist of a two-layer, two-component epoxy polymer in conjunction with natural or synthetic aggregates for a 3/8-inch total thickness. Refer to the approved products list for a list of pre-qualified polymer liquid binders.

Thin polymer overlays can be placed on new concrete once it has been fully cured and dried to an acceptable moisture content, which may be as soon as 21 days. However, cracks will develop in the concrete deck throughout the first couple of years in response to vehicular and environmental loads. As a result, the preferred time to place the overlay is after initial concrete cracking, which should occur within the first two years of a new deck. Placement after this time allows the overlay to seal existing cracks and may reduce reflective cracking in the overlay. This application window is not ideal for projects, since it will usually require an additional contract for the overlay application. As a result, it is recommended that decks be sealed for the first several years and then receive a thin polymer overlay.

Sufficient bond strength is critical in maximizing the overlay's service life. The bond strength can be reduced by poor surface preparations, traffic conditions, moisture, and distressed concrete. As a result, TPO's should be used based on the following restrictions:

- Recommended on decks with a NBI rating greater than 7 to help mitigate chloride infiltration. The deck should be in good condition with wearing surface distressed areas not exceeding 2% of the total deck area.

- Not recommended on decks that have been exposed to chlorides for more than 10 years old or with a NBI rating less than 7. These restrictions assume that significant chloride infiltration has already occurred. When a robust deck washing and sealing program has been used, TPO's may be placed on decks 10-15 years old with above average deck condition. Roadway traffic volume should also be a consideration for determining when to apply a TPO. As roadway volumes increase, it is assumed that chloride infiltration occurs significantly faster due to the increased application of deicing salts.

- Decks with an existing TPO may be considered for a TPO re-application provided that the previously discussed restrictions can be assumed to be satisfied. Generally, this assumes the existing overlay performed well over its expected service life and the effective deck exposure did not exceed 15 years, such that significant chloride infiltration has not occurred. If signification chloride infiltration is expected, a re-application would not be recommended.

- TPO's should not be placed on concrete decks or patches less than 21 days old or as required by WisDOT's or the supplier's specifications, whichever is more restrictive. Patch and crack repairs shall be compatible with the overlay material.

- Use of TPO's on the concrete approaches should be avoided. Slab-on-grade conditions may cause the overlay to fail prematurely due to moisture issues.

- Not recommended on decks with widespread cracking, large cracks (>0.04 in), or active cracks (e.g. longitudinal reflective cracks between PS box girders). These cracks are likely to reflect through the overlay, even when fully repaired.

Thin polymer overlays may be considered where friction needs to be restored or improved. In most cases, the two-layer polymer overlay system should be used as it will improve surface friction and protect the deck against future chloride infiltration. For situations requiring a high skid resistance, calcined bauxite or other alternative aggerates may be considered in lieu of natural or synthetic aggregates.

40.5.1.2 Low Slump Concrete Overlay

A low slump concrete overlay, also referred to as a concrete overlay, is expected to extend the service life of a bridge deck for 15 to 20 years. This system is comprised of low slump Grade E concrete and has a 1-1/2 inch minimum thickness. The overlay thickness can accommodate profile and cross-slope differences, but typically does not exceed 4-1/2 inches. Thicker overlays become increasingly unpractical due to load and cost implications.

Low slump Grade E concrete requires close adherence to the specification, including equipment, consolidation, and curing requirements. A properly cured concrete overlay will help limit cracks, but inevitably the concrete overlay will crack. After the concrete overlay has been placed, it is beneficial to seal cracks in the overlay to minimize deterioration of the underlying deck. The overlay may require crack sealing the following year and periodically thereafter.

On delaminated but structurally sound decks, a rehabilitation concrete overlay is often the only alternative to deck replacement. Typically, prior to placing the concrete overlay a minimum of 1" of existing deck surface is removed along with any unsound material and asphaltic patches.

> Rehabilitation concrete overlays are performed when significant distress of the wearing surface has occurred. If more than 25% of the wearing surface is distressed, an in-depth cost analysis should be performed to determine if a concrete overlay is cost effective verses a deck replacement.
>
> The quantity of distress on the underside of deck or slab should be negligible, less than 5%, indicating that the bottom mat of reinforcement steel is not significantly deteriorated. If significant quantities of distress are present under the deck, a deck replacement may be required in the future; an overlay at this time might not achieve full service life, but may be placed to provide a good riding surface until replacement.

If the structure has an existing overlay, the overlay condition should be evaluated in addition to the other previously discussed considerations. If the concrete deck remains structurally sound, it may be practical to remove an existing overlay and place a new overlay before replacing the entire deck. Prior to placing the concrete overlay, the existing overlay should be removed to at least the original deck surface. Additional surface milling may not be practical if the previous overlay included a milling operation.

40.5.1.3 Polyester Polymer Concrete Overlay

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

Prior to the placement of the PPC overlay, a high molecular weight methacrylate (HMWM) binder is placed on the prepared deck. This bonds the overlay to the deck, and it also serves to seal existing cracks in the deck. When the existing concrete is in good condition, PPC is effective at mitigating chloride penetration due to its impermeability.

The total thickness of a PPC overlay is typically 3/4” to 1”. While thicker overlays are possible, they are usually cost prohibitive. PPC can be placed at 3/4” thick as opposed to a typical 1 1/2” thick concrete overlay. This may help in situations where bridge ratings and/or profile adjustments are of concern.

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

- Deck wearing surface distress should not exceed 5% of the total deck area.
- Decks should have a NBI rating of 7 or greater and be less than 15 years old. Older decks may be considered when the existing deck has been protected by a thin polymer overlay or when chloride testing indicates acceptable chloride levels at the reinforcement. Decks exposed to chlorides, exceeding 10 years, should consider a ¾-inch minimum scarification to remove chlorides.
- Chloride contents at the reinforcement should not exceed 5 lbs/CY for decks with epoxy coated reinforcement. PPC overlays are not recommended on decks with uncoated top mat reinforcement.
- PPC overlays should not be placed on concrete decks or patches less than 28 days, unless approved otherwise. Patch and crack repairs shall be compatible with the overlay material.
- PPC shall not be used for structural repairs due to costs and performance concerns.

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

40.5.1.4 Polymer Modified Asphaltic Overlay

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

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

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

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

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

40.5.1.5 Asphaltic Overlay

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

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

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

40.5.1.6 Asphaltic Overlay with Waterproofing Membrane

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

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

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

40.5.1.7 Other Overlays

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

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

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

- Reinforced concrete overlays

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

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

40.5.2 Selection Considerations

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

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

(1) Estimated costs based on CY2017. Overlay costs includes minimum overlay thickness and overlay placement costs. Project costs includes all structure associated costs (joint repairs, deck repairs, surface preparations, minimum overlay thickness). Costs do not include traffic control costs or other costs not captured on structure costs.
(2) Requires approval
(3) Rapid set patch material requires approval. Concrete patch material requires 21-28 day cures.
(4) Not eligible for federal funds
(5) 1 to 3/4-inch milling recommended for decks exposed longer than 10 years and not previously milled
(6) Estimated durations based on the overlay placement time to the minimum time until traffic can to be placed on the overlay. This duration does not include time for deck preparations or staging considerations.

Table 40.5-1
Overlay Selection Considerations

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

Table 40.5-2

Overlay Advantages, Disadvantages, and Notes

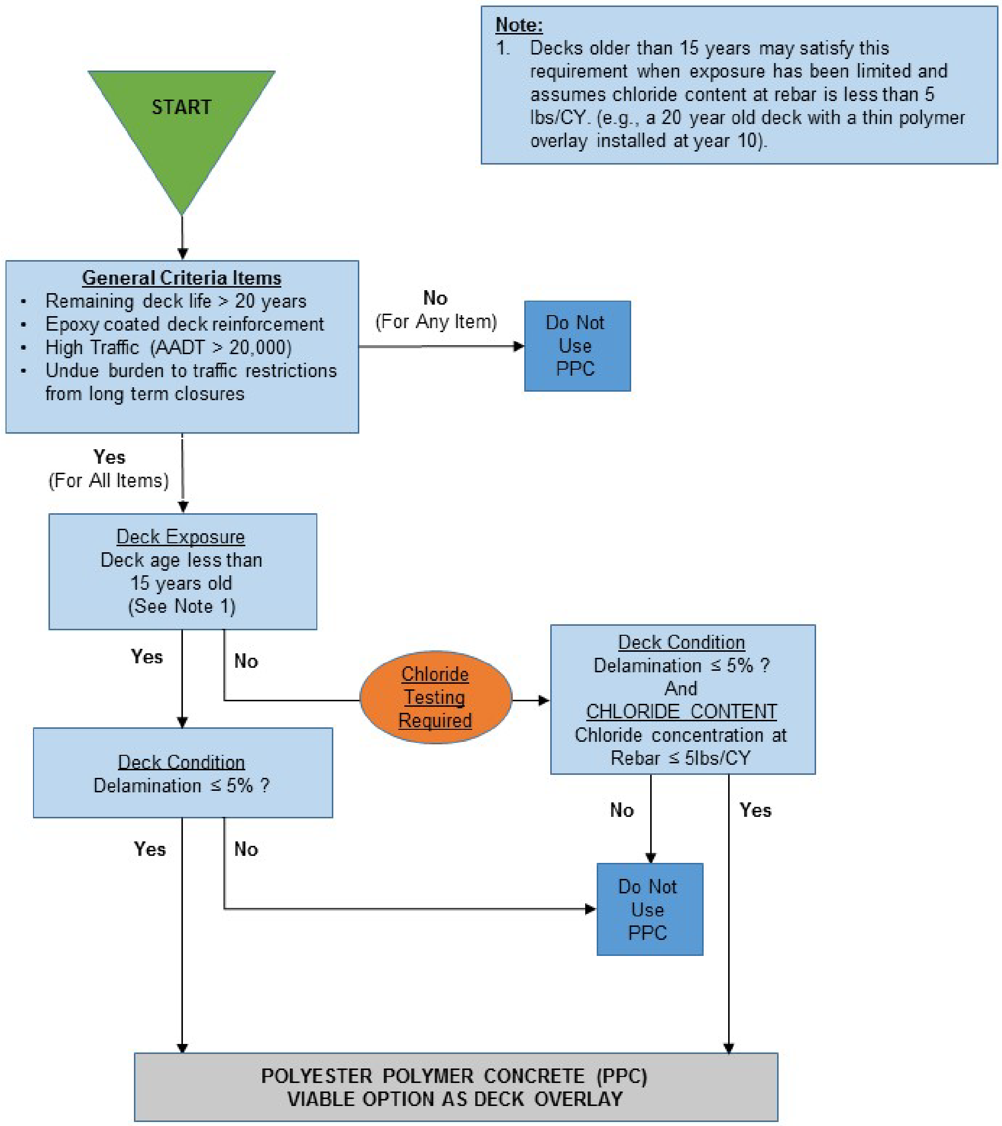

Figure 40.5-2

Polyester Polymer Concrete Overlay Usage Flowchart

40.5.3 Deck Assessment

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

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

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

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

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

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

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

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

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

40.5.4 Deck Preparations

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

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

Deck Repairs

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

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

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

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

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

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

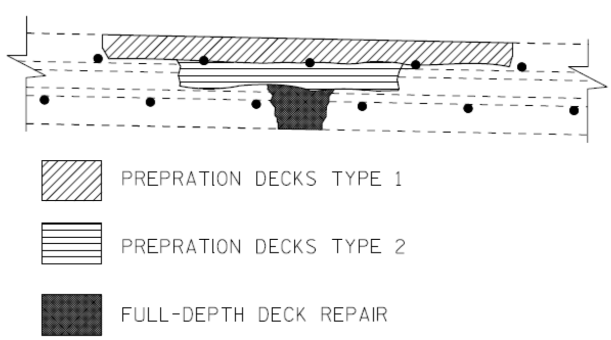

Figure 40.5-3
Deck Repairs

Deck Patches

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

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

Concrete Removal

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

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

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

Surface Removal and Surface Preparation

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

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

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

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

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

40.5.5 Preservation Techniques

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

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

For additional preservation techniques and information refer to the *Bridge Preservation Policy Guide.*

40.5.6 Other Considerations

- Bridges with Inventory Ratings less than HS10 after rehabilitation shall not be considered for overlays, unless approved by the Bureau of Structures Design Section.
- Inventory and Operating Ratings shall be provided on the bridge rehabilitation plans.
- Verify the desired transverse cross slope with the Regions as they may want to use current standards.

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

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

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

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

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

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

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

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

40.5.7 Past Bridge Deck Protective Systems

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

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

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

- High Performance Concrete (HPC) - Use of HPC has been limited to Mega Projects.
- Thin Polymer Overlays – Use of this overlay system is currently being used.
- Polyester Polymer Overlays – Use of this overlay system currently being used limitedly.
- Additional Concrete Cover – Use of additional clear cover (> 2 ½ inches) has been used on areas with high volume and high truck traffic.
- Stainless steel deck reinforcement – Use of stainless steel has been very limited.
- Fiber reinforce polymer (FRP) deck reinforcement - Use of FRP reinforcement has only be used for experimental purposes.

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

40.5.8 Railings and Parapets

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

40.6 Deck Replacements

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

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

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

Table 40.6-1
Condition Requirements for Deck Replacements

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

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

WisDOT policy item:

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

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

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

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

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

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

40.7 Rehabilitation Girder Sections

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

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

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

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

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

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

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

Table 40.7-1
Maximum Span Length vs. Girder Spacing

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

40.8 Widenings

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

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

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

For multi-columned piers, consider the cap connection to the existing cap as pinned and design the new portion to the latest LRFD criteria. The new column(s) are not required to meet **LRFD [3.6.5]** (600 kip loading) as a widening is considered rehabilitation. Abutments shall be widened to current LRFD criteria as well as the Standard for Abutment Widening.

For foundation support, use the most current method available.

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

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

40.9 Superstructure Replacements/Moved Girders (with Widening)

When steel girder bridges have girder spacings of 3' or less and require expensive redecking or deck widening; consider replacing the superstructure with a concrete slab.

When replacing a deck on a steel girder bridge, the cost of painting and structural rehab shall be considered to determine whether a deck or superstructure replacement is more cost effective (provided the substructure is sufficient to support the loading). Also, additional costs will occur if bearings, floor drains, expansion joints, railings, or other structural elements require replacement.

Approval is required from BOS for all superstructure replacement projects. In order for a superstructure replacement to be allowed, the substructure must meet the criteria outlined below. This justifies the cost of a new superstructure by ensuring a uniform level of reliability for the entire structure.

Evaluate the existing piers using current LRFD criteria. If an existing multi-columned pier has 3 or more columns, the 600 kip vehicular impact loading need not be considered if the pier is adjacent to a roadway with a design speed ≤ 40 mph. If the design speed is 45 mph or 50 mph, the 600 kip vehicular impact loading need not be considered if a minimum "vehicle protection" is provided as per FDM 11-35-1. For design speeds > 50 mph, all criteria as per 13.4.10 must be met.

Abutment bodies should be evaluated using Strength I loading.

Evaluate the pier or abutment piles, or bearing capacity of the soil/rock if on spread footings, utilizing Strength I loading. Contact the Bureau of Technical Services Geotechnical Engineering Unit for guidance regarding the factored resistance of the existing piles or bearing capacity of the soil/rock.

The superstructure shall be designed to current LRFD criteria.

40.10 Replacement of Impacted Girders

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

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

40.11 New Bridge Adjacent to Existing Bridge

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

The pier design shall be to current LRFD criteria, including the 600 kip impact load for the new bridge. It is not required to strengthen or protect the existing adjacent pier for the 600 kip impact load. However, it would be prudent to discuss with the Region the best course of action. If the Region wants to provide crash protection, it may be desirable to provide TL-5 barrier/crash wall protection for both structures, thus eliminating the need to design the new pier for the 600 kip impact load. The Region may also opt to provide typical barrier protection (< TL-5) to both sets of piers, in which case the design engineer would still be required to design the new pier for the 600 kip impact load. This last option is less expensive than providing TL-5 barrier to both structures. Aesthetics are also a consideration in the above choices.

40.12 Timber Abutments

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

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

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

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

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

40.13 Survey Report and Miscellaneous Items

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

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

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

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

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

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

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

Structure plans (using a sheet border with a #8 tab) are required for all structure rehabilitation projects. This includes work such as superstructure painting projects and all overlay projects, including polymer overlay projects. See Chapter 6-Plan Preparation guidance for plan minimum requirements.

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

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

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

40.14 Superstructure Inspection

40.14.1 Prestressed Girders

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

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

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

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

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

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

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

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

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

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

   or

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

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

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

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

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

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

4. Vertical misalignment in excess of the normal allowable.

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

40.14.2 Steel Beams

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

1. Replace the total beam,

2. Replace a section of the beam, or

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

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

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

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

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

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

40.15 Substructure Inspection

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

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

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

40.15.1 Hammerhead Pier Rehabilitation

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

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

1. Place a watertight expansion joint in the deck.

2. Consider whether bearing replacement is required.

3. Analyze the type of cap repair required.

    a. Clean off spalled concrete and place new concrete.

    b. Analyze capacity of bars still bonded to see if unbonded bars are needed. Use ultimate strength analysis.

    c. Consider repair method for serious loss of bar steel capacity.

        i. Add 6” of cover to cap. Add additional bar steel. Grout in U shaped stirrups around bars using standard anchor techniques.

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

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

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

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

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

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

2. Place wire mesh around shaft.

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

40.15.2 Bearings

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

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

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

40.16 Concrete Anchors for Rehabilitation

Concrete anchors are used to connect concrete elements with other structural or non-structural elements and can either be cast into concrete (cast-in-place anchors) or installed after concrete has hardened (post-installed anchors). This section discusses post installed anchors used on bridge rehabilitation projects. Note: this section is also applicable for several cases where post installed anchors may be allowed in new construction.

This section includes guidance based on the ACI 318-14 manual, hereafter referred to as ACI. (AASHTO currently does not have guidance for anchors.)

40.16.1 Concrete Anchor Type and Usage

Concrete anchors installed in hardened concrete, post-installed anchors, typically fall into two main groups – adhesive anchors and mechanical anchors. For mechanical anchors, subgroups include undercut anchors, expansion (torque-controlled or displacement controlled) anchors, and screw anchors.

Mechanical anchors are seldom used for bridge rehabilitations and current usage has been restricted due to the following concerns: anchor installation (hitting rebar, abandoning holes, and testing), the number of different anchor types, design requirements that are more restrictive than adhesive anchors, the ability to remove and reuse railings/fences, and the collection of salt water within the hole. Note: mechanical anchors may be considered when it has been determined cast-in-place anchors or through bolts are cost prohibitive, adhesive anchors are not recommended, and the above concerns for mechanical anchors have been addressed. See post-installed anchor usage restrictions for additional information.

An Approved Products List addresses some of the concerns for creep, shrinkage, and deterioration under load and freeze-thaw cycles for adhesives anchors. Bridge rehabilitations projects typically use adhesive anchors for abutment and pier widenings. Other bridge rehabilitation applications may also warrant the use of adhesive anchors when required to anchor into existing concrete. Refer to the Standards for several examples of anchoring into existing concrete.

In limited cases, post installed concrete anchors may be allowed for new construction. One application is the allowance for the contractor to use adhesive anchors in lieu of cast-in-place concrete anchors for attaching pedestrian railings/fencing. Refer to Chapter 30 Standards for pedestrian railings/fencing connections.

The following is a list of current usage restrictions for post installed anchors:

Usage Restrictions:

- Pier cap extensions for multi-columned piers require additional column(s) to be utilized. See Chapter 13 – Piers for structural modeling concepts regarding multi-columned piers.

- **Contact the Bureau of Structures if considering any extension of a hammerhead pier (without additional vertical support from an added column).**

- Adhesive anchors installed in the overhead or upwardly inclined position and/or under sustained tension loads shall not be used.

- The department has placed a moratorium on mechanical anchors. Usage is subject to prior-approval by the Bureau of Structures.

40.16.1.1 Adhesive Anchor Requirements

For adhesive anchors, there are two processes used to install the adhesive. One option uses a two-part adhesive that is mixed and poured into the drilled hole. The second option pumps a two-part adhesive into the hole by a dispenser which combines the two components at the nozzle just prior to entering the hole or within the hole. With either process, the hole must be properly cleaned and a sufficient amount of adhesive must be used so that the hole is completely filled with adhesive when the rebar or bolt is inserted. The adhesive bond stresses, as noted in Table 40.16 1, are determined by the 5 percent fractile of results of tests performed and evaluated according to ICC-ES AC308 or ACI 355.4.

The required minimum anchor spacing is 6 times the anchor diameter. The minimum edge distance is 6 times the anchor diameter. The maximum embedment depth for is 20 times the anchor diameter.

The manufacturer and product name of adhesive anchors used by the contractor must be on the Department's approved product list for "Concrete Adhesive Anchors".

Refer to the *Standard Specifications* for additional requirements.

40.16.1.2 Mechanical Anchor Requirements

The required minimum anchor spacing is 6 times the anchor diameter. The minimum edge distance is 10 times the anchor diameter. The minimum member is the great of the embedment depth plus 4 inches and 3/2 of the embedment depth. ***Mechanical anchors are currently not allowed.***

40.16.2 Concrete Anchor Reinforcement

Reinforcement used to transfer the full design load from the anchors into the structural member is considered anchor reinforcement. **ACI [17.4.2.9]** and **ACI [17.5.2.9]** provide guidance for designing anchor reinforcement. When anchor reinforcement is used, the design strength of the anchor reinforcement can be used in place of concrete breakout strength per 40.16.3 and 40.16.4. Reinforcement that acts to restrain the potential concrete breakout but is not designed to transfer the full design load is considered to be supplementary reinforcement.

Per **ACI [2.3],** concrete anchor steel is considered ductile if the tensile test elongation is at least 14 percent and reduction in area is at least 30 percent. Additionally, steel meeting the requirements of ASTM A307 is considered ductile. Steel that does not meet these requirements is considered brittle. Rebar used as anchor steel is considered ductile.

40.16.3 Concrete Anchor Tensile Capacity

Concrete anchors in tension fail in one of four ways: steel tensile rupture, concrete breakout, pullout strength of anchors in tension, or adhesive bond. The pullout strength of anchors in tension only applies to mechanical anchors and the adhesive bond only applies to adhesive anchors. Figure 40.16-1 shows the concrete breakout failure mechanism for anchors in tension.

The minimum pullout capacity (Nominal Tensile Resistance) of a single concrete anchor is determined according to this section; however, this value is only specified on the plan for mechanical anchors. The minimum pullout capacity is not specified on the plan for adhesive anchors because the anchors must be designed to meet the minimum bond stresses as noted in Table 40.16-1. If additional capacity is required, a more refined analysis (i.e., anchor group analysis) per the current version of ACI 318-14 Chapter 17 is allowable, which may yield higher capacities.

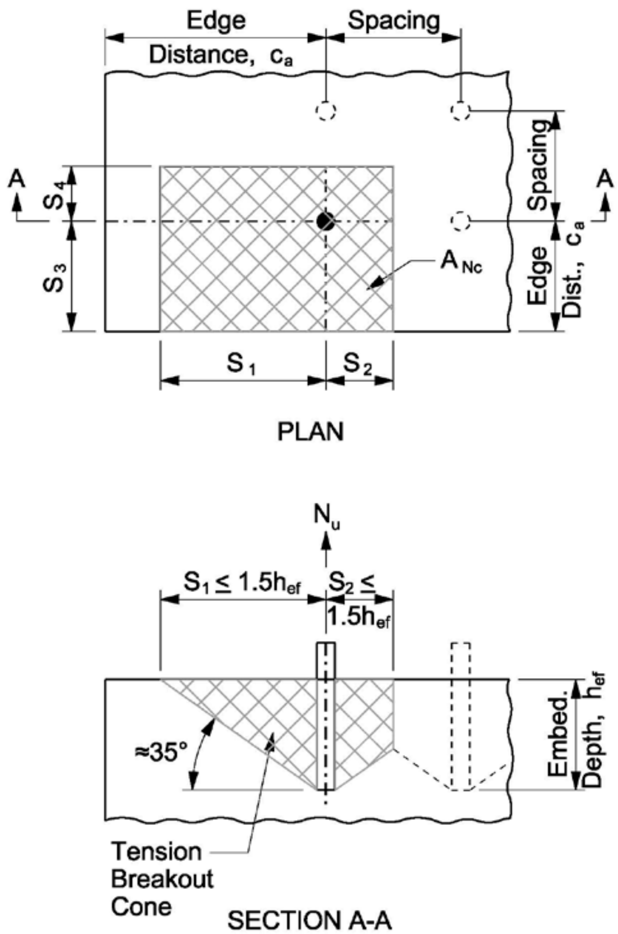

Figure 40.16-1
Concrete Breakout of Concrete Anchors in Tension

The projected concrete breakout area, A_{Nc}, shown in Figure 40.16-1 is limited in each direction by S_i:

S_i = Minimum of:

1. 1.5 times the embedment depth (h_{ef}),
2. Half of the spacing to the next anchor in tension, or
3. The edge distance (c_a) (in).

Figure 40.16-2 shows the bond failure mechanism for concrete adhesive anchors in tension.

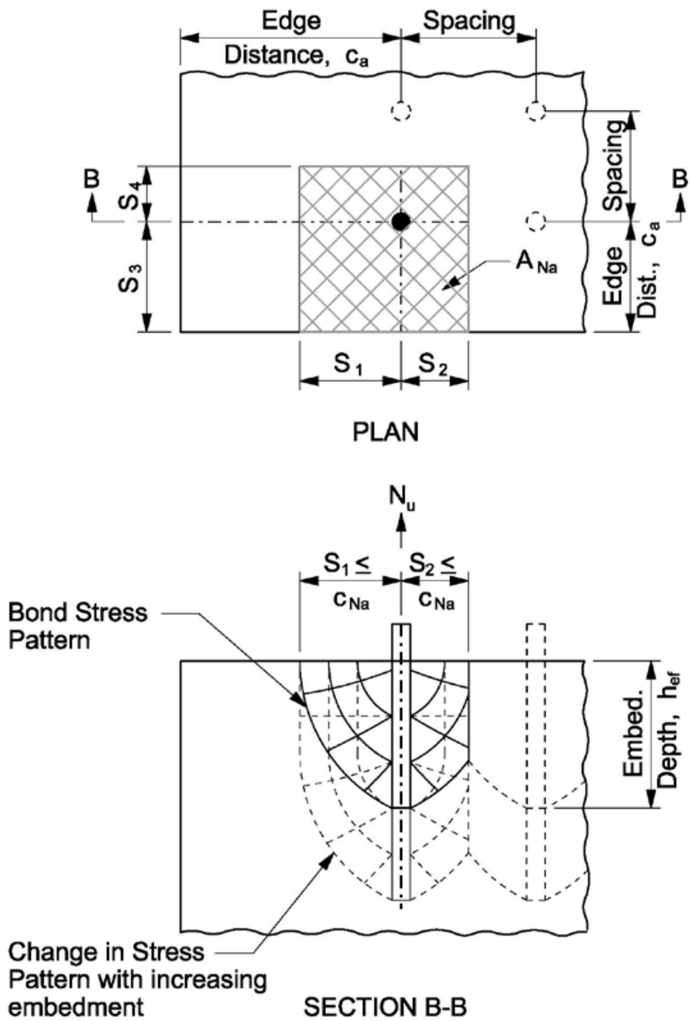

Figure 40.16-2
Bond Failure of Concrete Adhesive Anchors in Tension

The projected influence area of a single adhesive anchor, A_{Na}, is shown in Figure 40.16-2. Unlike the concrete breakout area, it is not affected by the embedment depth of the anchor. A_{Na} is limited in each direction by S_i:

S_i = Minimum of:

1. $c_{Na} = 10 d_a \sqrt{\frac{\tau_{uncr}}{1100}}$,

2. Half of the spacing to the next anchor in tension, or

3. The edge distance (c_a) (in).

Anchor Size, d_a	Adhesive Anchors			
	Dry Concrete		Water-Saturated Concrete	
	Min. Bond Stress, τ_{uncr} (psi)	Min. Bond Stress, τ_{cr} (psi)	Min. Bond Stress, τ_{uncr} (psi)	Min. Bond Stress, τ_{cr} (psi)
#4 or 1/2"	990	460	370	280
#5 or 5/8"	970	460	510	390
#6 or 3/4"	950	490	500	410
#7 or 7/8"	930	490	490	340
#8 or 1"	770	490	600	340

Table 40.16-1
Tension Design Table for Concrete Anchors

The minimum bond stress values for adhesive anchors in Table 40.16-1 are based on the Approved Products List for "Concrete Adhesive Anchors". The designer shall determine whether the concrete adhesive anchors are to be utilized in dry concrete (i.e., rehabilitation locations where concrete is fully cured, etc.) or water-saturated concrete (i.e., new bridge decks, box culverts, etc.) and shall design the anchors accordingly.

The factored tension force on each anchor, N_u, must be less than or equal to the factored tensile resistance, N_r. For mechanical anchors:

$$N_r = \phi_{ts} N_{sa} \le \phi_{tc} N_{cb} \le \phi_{tc} N_{pn}$$

In which:

ϕ_{ts} = Strength reduction factor for anchors in concrete**, ACI [17.3.3]**
= 0.65 for brittle steel as defined in 40.16.1.1
= 0.75 for ductile steel as defined in 40.16.1.1

N_{sa} = Nominal steel strength of anchor in tension, **ACI [17.4.1.2]**
= $A_{se,N} f_{uta}$

$A_{se,N}$ = Effective cross-sectional area of anchor in tension (in^2)

f_{uta} = Specified tensile strength of anchor steel (psi)

$\leq 1.9f_{ya}$

$\leq$ 125 ksi

f_{ya} = Specified yield strength of anchor steel (psi)

ϕ_{tc} = Strength reduction factor for anchors in concrete
= 0.65 for anchors without supplementary reinforcement per 40.16.2
= 0.75 for anchors with supplementary reinforcement per 40.16.2

N_{cb} = Nominal concrete breakout strength in tension, **ACI [17.4.2.1]**

$$= \frac{A_{Nc}}{9(h_{ef})^2}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$$

A_{Nc} = Projected concrete failure area of a single anchor, see Figure 40.16-1
= $(S_1 + S_2)(S_3 + S_4)$

h_{ef} = Effective embedment depth of anchor per Table 40.16-1. May be reduced per **ACI [17.4.2.3]** when anchor is located near three or more edges.

$\psi_{ed,N}$ = Modification factor for tensile strength based on proximity to edges of concrete member, **ACI [17.4.2.5]**
= 1.0 if $c_{a,min} \geq 1.5h_{ef}$

$$= 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}} \text{ if } c_{a,min} < 1.5h_{ef}$$

$c_{a,min}$ = Minimum edge distance from center of anchor shaft to the edge of concrete, see Figure 40.16-1 (in)

$\psi_{c,N}$ = Modification factor for tensile strength of anchors based on the presence or absence of cracks in concrete, **ACI [17.4.2.6]**
= 1.0 when post-installed anchors are located in a region of a concrete member where analysis indicates cracking at service load levels
= 1.4 when post-installed anchors are located in a region of a concrete member where analysis indicates no cracking at service load levels

$\psi_{cp,N}$ = Modification factor for post-installed anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation, **ACI [17.4.2.7]**

= 1.0 if $c_{a,min} \geq c_{ac}$

= $\frac{c_{a,min}}{c_{ac}} \geq \frac{1.5h_{ef}}{c_{ac}}$ if $c_{a,min} < c_{ac}$

c_{ac} = Critical edge distance (in)

= $4.0h_{ef}$

N_b = Concrete breakout strength of a single anchor in tension in uncracked concrete, **ACI [17.4.2.2]**

= $0.538\sqrt{f'_c}(h_{ef})^{1.5}$ (kips)

N_{pn} = Nominal pullout strength of a single anchor in tension, **ACI [17.4.3.1]**

= $\psi_{c,P}N_p$

$\psi_{c,P}$ = Modification factor for pullout strength of anchors based on the presence or absence of cracks in concrete, **ACI [17.4.3.6]**

= 1.4 where analysis indicates no cracking at service load levels

= 1.0 where analysis indicates cracking at service load levels

N_p = Nominal pullout strength of a single anchor in tension based on the 5 percent fractile of results of tests performed and evaluated according to ICC-ES AC193 / ACI 355.2

<u>For adhesive anchors:</u>

$$N_r = \phi_{ts}N_{sa} \leq \phi_{tc}N_{cb} \leq \phi_{tc}N_a$$

In which:

N_{cb} = Nominal concrete breakout strength in tension, **ACI [17.4.2.1]**

= $\frac{A_{Nc}}{9(h_{ef})^2}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$

h_{ef} = Effective embedment depth of anchor. May be reduced per **ACI [17.4.2.3]** when anchor is located near three or more edges.

≤ $20d_a$ (in)

d_a = Outside diameter of anchor (in)

$\psi_{cp,N}$ = Modification factor for post-installed anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation, **ACI [17.4.2.7]**

= 1.0 if $c_{a,min} \geq c_{ac}$

= $\frac{c_{a,min}}{c_{ac}} \geq \frac{1.5h_{ef}}{c_{ac}}$ if $c_{a,min} < c_{ac}$

$c_{a,min}$ = Minimum edge distance from center of anchor shaft to the edge of concrete, see Figure 40.16-1 or Figure 40.16-2 (in)

c_{ac} = Critical edge distance (in)

= $2.0h_{ef}$

N_a = Nominal bond strength of a single anchor in tension, **ACI [17.4.5.1]**

= $\frac{A_{Na}}{4c_{Na}^{\ 2}}\psi_{ed,Na}\psi_{cp,Na}N_{ba}$

A_{Na} = Projected influence area of a single adhesive anchor, see Figure 40.16-2

= $(S_1 + S_2)(S_3 + S_4)$

$\psi_{ed,Na}$ = Modification factor for tensile strength of adhesive anchors based on the proximity to edges of concrete member, **ACI [17.4.5.4]**

= 1.0 if $c_{a,min} \geq c_{Na}$

= $0.7 + 0.3\frac{c_{a,min}}{c_{Na}}$ if $c_{a,min} < c_{Na}$

c_{Na} = Projected distance from center of anchor shaft on one side of the anchor required to develop the full bond strength of a single adhesive anchor

= $10d_a\sqrt{\frac{\tau_{uncr}}{1100}}$ (in)

τ_{uncr} = Characteristic bond stress of adhesive anchor in uncracked concrete, see Table 40.16-1

$\psi_{cp,Na}$ = Modification factor for pullout strength of adhesive anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation, **ACI [17.4.5.5]**

= 1.0 if $c_{a,min} \geq c_{ac}$

$$= \frac{c_{a,min}}{c_{ac}} \geq \frac{c_{Na}}{c_{ac}} \text{ if } c_{a,min} < c_{ac}$$

N_{ba} = Bond strength in tension of a single adhesive anchor, **ACI [17.4.5.2]**

= $\tau_{cr} \pi d_a h_{ef}$

τ_{cr} = Characteristic bond stress of adhesive anchor in cracked concrete, see Table 40.16-1

Note: Where analysis indicates cracking at service load levels, adhesive anchors shall be qualified for use in cracked concrete in accordance with ICC-ES AC308 / ACI 355.4. For adhesive anchors located in a region of a concrete member where analysis indicates no cracking at service load levels, τ_{uncr} shall be permitted to be used in place of τ_{cr}.

In addition to the checks listed above for all adhesive anchors, the factored sustained tensile force must be less than or equal to the factored sustained tensile resistance per **ACI [17.3.1.2]**:

$0.50\ \phi_{tc} N_{ba} \geq N_{ua,s}$

40.16.4 Concrete Anchor Shear Capacity

Concrete anchors in shear fail in one of three ways: steel shear rupture, concrete breakout, or concrete pryout. Figure 40.16-3 shows the concrete breakout failure mechanism for anchors in shear.

The projected concrete breakout area, A_{Vc}, shown in Figure 40.16-3 is limited vertically by H, and in both horizontal directions by S_i:

H = Minimum of:

1. The member depth (h_a) or
2. 1.5 times the edge distance (c_{a1}) (in).

S_i = Minimum of:

1. Half the anchor spacing (S),
2. The perpendicular edge distance (c_{a2}), or
3. 1.5 times the edge distance (c_{a1}) (in).

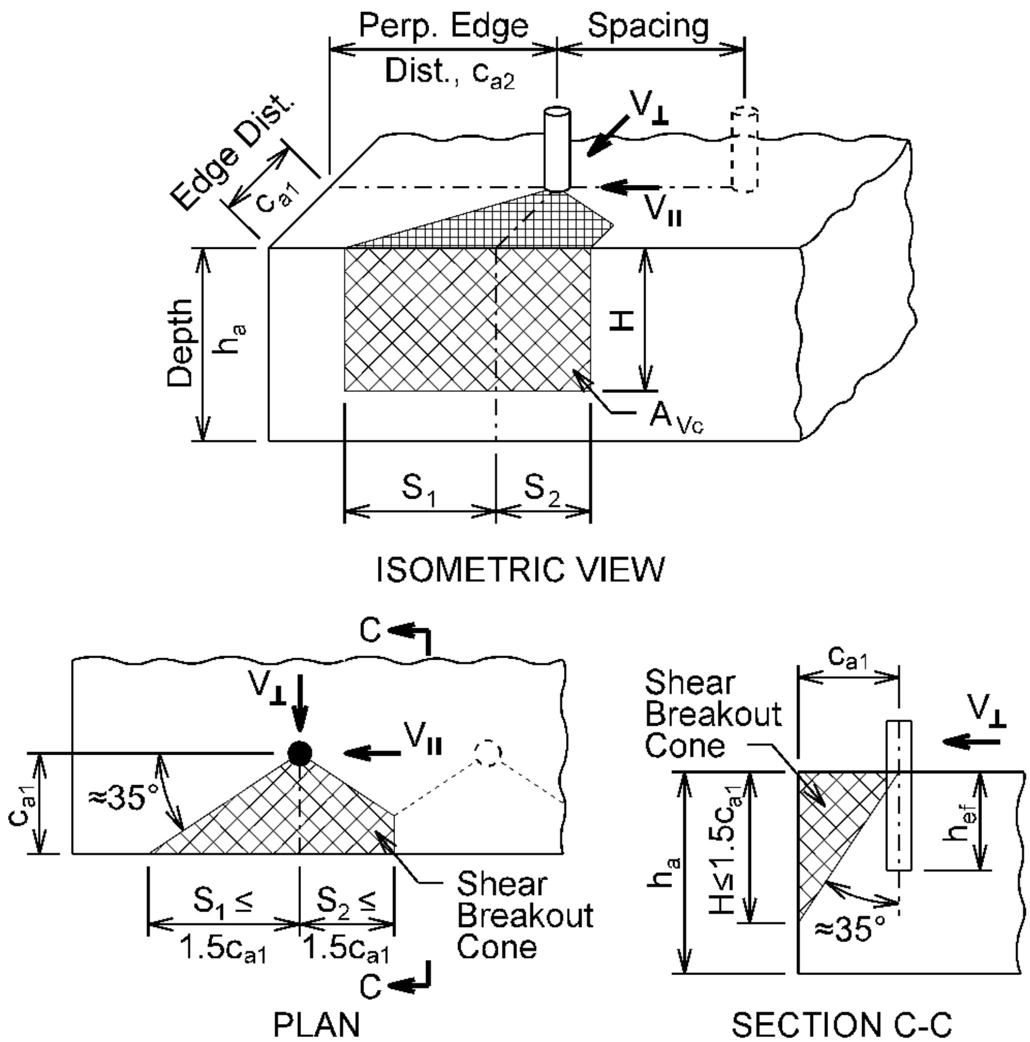

Figure 40.16-3

Concrete Breakout of Concrete Anchors in Shear

If the shear is applied to more than one row of anchors as shown in Figure 40.16-4, the shear capacity must be checked for the worst of the three cases. If the row spacing, SP, is at least equal to the distance from the concrete edge to the front anchor, E1, check both Case 1 and Case 2. In Case 1, the front anchor is checked with the shear load evenly distributed between the rows of anchors. In Case 2, the back anchor is checked for the full shear load. If the row spacing, SP, is less than the distance from the concrete edge to the front anchor, E1, then check Case 3. In case 3, the front anchor is checked for the full shear load. If the anchors are welded to an attachment to evenly distribute the force to all anchors, only Case 2 needs to be checked.

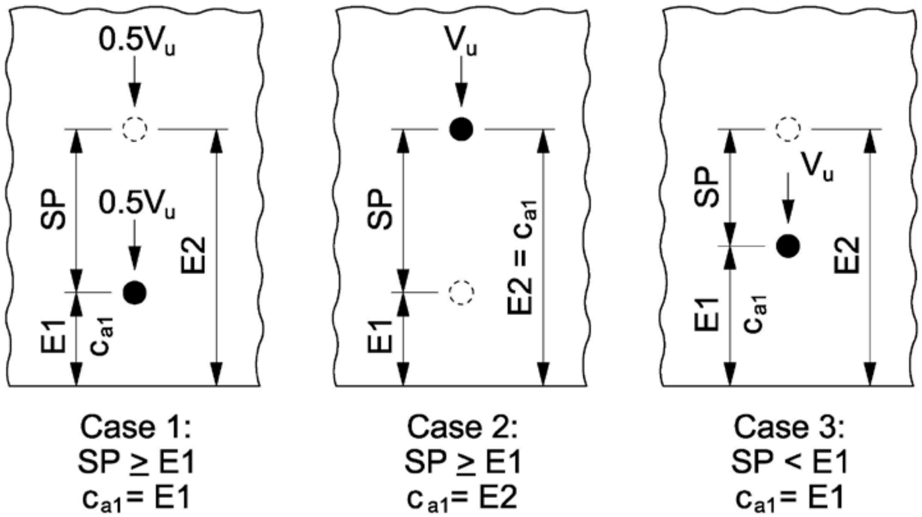

Figure 40.16-4
Concrete Anchor Shear Force Cases

The factored shear force on each anchor, V_u, must be less than or equal to the factored shear resistance, V_r. For mechanical and adhesive anchors:

$$V_r = \phi_{vs} V_{sa} \leq \phi_{vc} V_{cb} \leq \phi_{vp} V_{cp}$$

In which:

ϕ_{vs} = Strength reduction factor for anchors in concrete, **ACI [17.3.3]**
= 0.60 for brittle steel as defined in 40.16.1.1
= 0.65 for ductile steel as defined in 40.16.1.1

V_{sa} = Nominal steel strength of anchor in shear, **ACI [17.5.1.2]**
= $0.6 A_{se,V} f_{uta}$

$A_{se,V}$ = Effective cross-sectional area of anchor in shear (in^2)

ϕ_{vc} = Strength reduction factor for anchors in concrete, **ACI [17.3.3]**
= 0.70 for anchors without supplementary reinforcement per 40.16.2
= 0.75 for anchors with supplementary reinforcement per 40.16.2

V_{cb} = Nominal concrete breakout strength in shear, **ACI [17.5.2.1]**
= $\dfrac{A_{Vc}}{4.5(c_{a1})^2} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{p,V} V_b$

A_{Vc} = Projected area of the concrete failure surface on the side of the concrete member at its edge for a single anchor, see Figure 40.16-3
= $H(S_1 + S_2)$

c_{a1} = Distance from the center of anchor shaft to the edge of concrete in the direction of the applied shear, see Figure 40.16-3 and Figure 40.16-4 (in)

$\psi_{ed,V}$ = Modification factor for shear strength of anchors based on proximity to edges of concrete member, **ACI [17.5.2.6]**
= 1.0 if $c_{a2} \geq 1.5c_{a1}$ (perpendicular shear)
= $0.7 + 0.3\dfrac{c_{a2}}{1.5c_{a1}}$ if $c_{a2} < 1.5c_{a1}$ (perpendicular shear)
= 1.0 (parallel shear)

c_{a2} = Distance from the center of anchor shaft to the edge of concrete in the direction perpendicular to c_{a1}, see Figure 40.16-3 (in)

$\psi_{c,V}$ = Modification factor for shear strength of anchors based on the presence or absence of cracks in concrete and the presence or absence of supplementary reinforcement, **ACI [17.5.2.7]**
= 1.4 for anchors located in a region of a concrete member where analysis indicates no cracking at service load levels
= 1.0 for anchors located in a region of a concrete member where analysis indicates cracking at service load levels without supplementary reinforcement per **Error! Reference source not found.** or with edge reinforcement smaller than a No. 4 bar
= 1.2 for anchors located in a region of a concrete member where analysis indicates cracking at service load levels with reinforcement of a No. 4 bar or greater between the anchor and the edge
= 1.4 for anchors located in a region of a concrete member where analysis indicates cracking at service load levels with reinforcement of a No. 4 bar or greater between the anchor and the edge, and with the reinforcement enclosed within stirrups spaced at no more than 4 inches

$\psi_{h,V}$ = Modification factor for shear strength of anchors located in concrete members with $h_a < 1.5_{ca1}$, **ACI [17.5.2.8]**
= $\sqrt{\dfrac{1.5c_{a1}}{h_a}} \geq 1.0$

h_a = Concrete member thickness in which anchor is located measured parallel to anchor axis, see Figure 40.16-3 (in)

$\psi_{p,V}$ = Modification factor for shear strength of anchors based on loading direction, **ACI [17.5]**
= 1.0 for shear perpendicular to the concrete edge, see Figure 40.16-3
= 2.0 for shear parallel to the concrete edge, see Figure 40.16-3

V_b = Concrete breakout strength of a single anchor in shear in cracked concrete, per **ACI [17.5.2.2]** , shall be the smaller of:

$$[7(\frac{l_e}{d_a})^{0.2}\sqrt{d_a}]\sqrt{f'_c}(c_{a1})^{1.5} \text{ (lb)}$$

Where:
$l_e = h_{ef} \le 8d_a$
d_a = Outside diameter of anchor (in)
f'_c = Specified compressive strength of concrete (psi)

and

$$9\sqrt{f'_c}(c_{a1})^{1.5}$$

ϕ_{vp} = Strength reduction factor for anchors in concrete
= 0.65 for anchors without supplementary reinforcement per 40.16.2
= 0.75 for anchors with supplementary reinforcement per 40.16.2

V_{cp} = Nominal concrete pryout strength of a single anchor, **ACI [17.5.3.1]**
= $2.0N_{cp}$

Note: The equation above is based on $h_{ef} \ge 2.5$ in. All concrete anchors must meet this requirement.

N_{cp} = Nominal concrete pryout strength of an anchor taken as the lesser of:

mechanical anchors: $\dfrac{A_{Nc}}{9(h_{ef})^2}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$

adhesive anchors: $\dfrac{A_{Na}}{4(c_{Na})^2}\Psi_{ed,Na}\Psi_{cp,Na}N_{ba}$

and

$$\frac{A_{Nc}}{9(h_{ef})^2}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$$

For shear in two directions, check both the parallel and the perpendicular shear capacity. For shear on an anchor near a corner, check the shear capacity for both edges and use the minimum.

40.16.5 Interaction of Tension and Shear

For anchors that are subjected to tension and shear, interaction equations must be checked per **ACI [17.6]**.

If $\frac{V_{ua}}{\phi V_n} \leq 0.2$ for the governing strength in shear, then the full strength in tension is permitted: $\phi N_n \geq N_{ua}$. If $\frac{N_{ua}}{\phi N_n} \leq 0.2$ for the governing strength in tension, then the full strength in shear is permitted: $\phi V_n \geq V_{ua}$. If $\frac{V_{ua}}{\phi V_n} > 0.2$ for the governing strength in shear and $\frac{N_{ua}}{\phi N_n} > 0.2$ for the governing strength in tension, then:

$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2$$

40.16.6 Plan Preparation

The required minimum pullout capacity (as stated on the plans) is equal to the Nominal Tensile Resistance as determined in 40.16.3.

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

Adhesive anchors located in uncracked concrete:

ADHESIVE ANCHORS X/X-INCH (or No. X BAR). EMBED XX" IN CONCRETE. *(Illustrative only, values must be calculated depending on the specific situation).*

Adhesive anchors located in cracked concrete:

ADHESIVE ANCHORS X/X-INCH (or No. X BAR). EMBED XX" IN CONCRETE. ANCHORS SHALL BE APPROVED FOR USE IN CRACKED CONCRETE. *(Illustrative only, values must be calculated depending on the specific situation).*

When using anchors to anchor bolts or studs, the bolt, nut, and washer or the stud as detailed on the plans is included in the bid item “Adhesive Anchors _-Inch”.

For anchors using rebar, the rebar should be listed in the “Bill of Bars” and paid for under the bid item “Bar Steel Reinforcement HS Coated Structures”.

When adhesive anchors are used as an alternative anchorage the following note should be included in the plans:

> ADHESIVE ANCHORS SHALL CONFORM TO SECTION 502.2.12 OF THE STANDARD SPECIFICATION. *(Note only applicable when the bid item Adhesive Anchor is not used).*

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

40.17 Plan Details

1. Excavation for Structures on Overlays

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

   > Any excavation required to complete the overlay or paving block at the abutments is to be considered incidental to the bid item "(insert applicable bid item)".

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

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

3. On structure deck overlay projects:

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

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

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

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

5. Total Estimated Quantities

   The Region should provide the designer with a Rehabilitation Structure Survey Report that provides a complete description of the rehabilitation and estimated quantities. Contact the Region for clarifications on the scope of work.

   Additional items:

- Provide deck survey outlining areas of distress (if available). These plans will serve as documentation for future rehabilitations.

- Distressed areas should be representative of the surveyed areas of distress. Actual repairs will likely be larger than the reported values while removing all unsound materials.

- Provide Preparation Deck Type 1 & 2 and Full-Depth Repair estimates for areas of distress.

- Coordinate asphaltic materials with the Region and roadway designers.

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

40.18 Retrofit of Steel Bridges

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

40.18.1 Flexible Connections

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

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

40.18.2 Rigid Connections

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

Other partial constraint details are:

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

The solution is to create spaces large enough (approximately 1/4” or more) for more material to flex thus reducing the concentration of stress. For gusset connection plates provide a larger gap than 1/4” and no intersecting welds. For existing conditions it may be necessary to drill holes at high stress concentrations. For new conditions it would be better to design a rigid connection and attach to the flange rather than the web. For certain situations a fillet weld should be used over a partial penetration weld to allow slight movement.

40.19 Reinforcing Steel for Deck Slabs on Girders for Deck Replacements

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

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

Table 40.19-1

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

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

40.20 Fiber Reinforced Polymer (FRP)

40.20.1 Introduction

Fiber reinforced polymer (FRP) material is a composite composed of fibers encased in a polymer matrix. The fibers provide tensile strength while the resin protects the fibers and transfers load between them. FRP can be used to repair or to retrofit bridges. Repair is often defined as returning a member to its original condition after damage or deterioration while retrofitting refers to increasing the capacity of a member beyond its original capacity.

For plan preparations, FRP repairs and retrofits are categorized as either structural strengthening or non-structural protection. Contact the Bureau of Structures Design Section for current Special Provisions and for other FRP considerations.

40.20.2 Design Guidelines

While there is no code document for the design of FRP repairs and retrofits, there are two nationally recognized design guidelines: the *Guide Specification for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements* (14.) hereinafter referred to as the AASHTO FRP Guide, and the *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures ACI 440.2R-08 (15.)* hereinafter referred to as the ACI FRP Guide.

Note: BOS has been evaluating the design methodologies found in the AASHTO FRP Guide and ACI RFP Guide. Noticeable differences between the guides warrants further investigation, with input from industry representation. FRP repairs and retrofits shall be in accordance with the applicable Special Provisions.

40.20.3 Applicability

Not all concrete structures can be retrofitted or repaired using FRP. Most FRP research has been conducted on normal sized members, therefore many of the design equations cannot be used with exceptionally large or deep members. Additionally, members with disturbed regions (D-regions) such as deep beams and corbels are outside of the scope of many design equations.

The structure must have some amount of load carrying capacity prior to the installation of the FRP. Due to the potential for premature debonding, FRP cannot be counted on to carry service loads; it may only be used increase the ultimate capacity of the structure for strength and extreme event load cases. The unrepaired or unretrofitted structure be able to carry the service dead and live loads:

$$R_r \geq \eta_i[(DC + DW) + (LL + IM)]$$

Where:

R_r = factored resistance computed in accordance with AASHTO LRFD Section 5

η_i = load modifier = 1.0

DC = force effects due to components and attachments

DW = force effects due to wear surfaces and utilities

LL = force effects due to live load

IM = force effects due to dynamic load allowance

If capacity is added in flexure to accommodate increased loads, the shear capacity of the member must be checked to ensure that it is still sufficient for the new loading. For non-structural FRP applications, applicability checks may not be required.

40.20.4 Materials

A typical FRP system consists of a primer, fibers, resin, bonding material (either additional resin or an adhesive), and a protective coating. FRP is specified in terms of the types of fiber and resin, the number of layers, the fiber orientation and the geometry. FRP is sold as a system and all materials used should be from the same system.

40.20.4.1 Fibers

The most common types of fiber used for bridge repairs are glass and carbon. Glass fibers are not as stiff or as strong as carbon, but they are much less expensive. Unless there is reason to do otherwise, it is recommended that glass fibers be used for corrosion protection and spall control. Carbon fibers should be used for strengthening and crack control.

Carbon fibers cannot be used where the FRP comes into contact with steel out of concerns for galvanic corrosion due to the highly conductive nature of carbon fibers. For applications where galvanic corrosion is a concern, glass fibers may be used, even in structural applications.

Often, FRP is requested by the region to provide column confinement. The engineer must determine if the requested confinement is true confinement where the FRP puts the column into triaxial compression to increase the capacity and ductility, or if the FRP is confining a patch from spalling off. In the case of true confinement (which is very rare in Wisconsin), carbon fibers should be used and the repair requires structural design. For spall control, glass fibers are acceptable and the repair is considered non-structural.

40.20.4.2 Coatings

After the FRP has been installed and fully cured, a protective coating is applied to the entire system. A protective coating is needed to protect against ultraviolet degradation and can also provide resistance to abrasion, wear, and chemicals. In situations where the FRP is submerged in water, inert protective coatings can help prevent compounds in the FRP from leaching into the water, mitigating environmental impacts.

Protective coatings can be made from different materials depending on the desired coating characteristics. Common coating types include vinyl ester, urethane, epoxy, cementitious, and acrylic. Acrylic coatings are generally the least expensive and easiest to apply, though they may also be less durable. If no coating type is specified, it is likely that the manufacturer will provide an acrylic coating.

For shorter term repairs, acrylic coatings are sufficient, but longer repairs should consider other coating types such as epoxy. Any coating used must be compatible with the FRP system selected by the contractor.

40.20.4.3 Anchors

The bond between the FRP and the concrete is the most critical component of an FRP installation and debonding is the most common FRP failure mode. Certain FRP configurations use anchors to increase the attachment of the FRP and attempt to delay or prevent debonding. These anchors can consist of near surface mounted bars, fiber anchors, additional FRP strips, or mechanical anchors such as bolts. It is permitted to use additional U-wrap strips to anchor flexural FRP, but the use of additional longitudinal strips to anchor shear FRP is prohibited. The use of additional U-wrap strips for flexural anchorage is required in some instances.

Because neither design guide requires anchorage or provides information as to what constitutes anchorage, it is left to the discretion of the designer to determine if anchorage should be used and in what quantities. The use of anchors is highly encouraged, particularly for shear applications and in situations where there is increased potential for debonding such as reentrant corners.

Specifying anchors will add cost to the repair, but these costs may be offset by increased capacity accorded to anchored systems in shear. The additional costs can also be justified if debonding is a concern. If the designer chooses to use anchors, anchors should be shown on plans, but the design of the anchors is left to the manufacturer.

40.20.5 Flexure

Flexural FRP is applied along the tension face of the member, where it acts as additional tension reinforcement. The fibers should be oriented along the length of the member.

40.20.5.1 Pre-Design Checks

If the design of the FRP will be provided by the contractor or their consultant, the engineer must still perform certain checks before specifying FRP in the plans. For flexure, the engineer must check that the structure has sufficient capacity to carry service loads without additional capacity from the FRP, as discussed in 40.20.3.

40.20.5.2 Composite Action

Composite action of the deck slab can be considered when designing flexural FRP repairs for girders, provided that the deck was designed to be composite. If composite action is considered, composite section properties must be computed. These properties should be substituted into the design equations presented in this section. Accounting for composite action will increase the capacity provided by the FRP.

40.20.5.3 Pre-Existing Substrate Strain

Unless all loads are removed from the member receiving FRP (including self-weight), there will be strain present in the concrete when the FRP is applied. This initial or pre-existing substrate strain ε_{bi} is computed through elastic analysis. All loads supported by the member during FRP installation should be considered and cracked section properties should be considered if necessary.

40.20.5.4 Deflection and Crack Control

Conduct standard LRFD serviceability checks for deflection and crack control while accounting for the contribution of the FRP. Because both the FRP and the concrete will be in the elastic zone at service levels, standard elastic analysis can be used to determine stresses and strains. Transformed section analysis can be used to transform the FRP into an equivalent area of concrete for the purposes of analysis. The condition of the member determines if the cracked or uncracked section properties should be used in computations.

40.20.6 Shear

In shear repair/retrofitting applications, the FRP acts essentially as external stirrups. The FRP wrap is applied with the fibers running transverse to the member.

40.20.6.1 Pre-Design Checks

If the design of the FRP will be provided by the contractor or their consultant, the engineer must still perform certain checks before specifying FRP in the plans. For shear, the engineer must check that the structure has sufficient capacity to carry service loads without additional capacity from the FRP, as discussed in 40.20.3.

Additionally, the engineer must ensure that the amount of FRP capacity required does not exceed the maximum allowable shear reinforcement. <u>It is important to note that the FRP capacity listed on the plans will be a factored capacity</u>, while the maximum allowable shear reinforcement check is for an unfactored capacity. Strength reduction factors must be incorporated to make a proper comparison.

If the FRP capacity is close to the maximum allowed, the designer must take care to ensure that a design is feasible. The capacity provided by FRP depends on the number of FRP layers, with each additional layer providing a discrete increase in capacity. There may be a situation where n layers does not provide enough capacity, but n+1 layers provides too much capacity and violates the maximum allowable shear reinforcement criteria. Changes in spacing of the wraps may help decrease the capacity provided by the FRP.

Example problems in shear can be found in the appendices of NCHRP Report 655 (16) and potential shear wrapping configurations can be found in NCHRP Report 678 (17).

40.21 References

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

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

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

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

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

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

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

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

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

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

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

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

13. *Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary,* 2011.

14. AASHTO. 2012. *Guide Specifications for the Design of Bonded FRP Systems for Repair and Strenghening of Concrete Bridge Elements.* Washington D.C.: American Assocation of State Highway and Transportation Officials, 2012

15. ACI. 2008. *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures.* Farmington Hills MI: American Concrete Institute, 2008. ACI 440.2R-08.

16. Zureick, A.-H., et al. 2010. *Recommended Guide Specification for the Design of Externally Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements.* Washington D.C.: Transportation Research Board, 2010. NCHRP Report 655

17. Belarbi, A., et al. 2011. *Design of FRP Systems for Strengthening Concrete Girders in Shear.* Washington D.C.: Transportation Research Board, 2011. NCHRP Report 678.

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E45-2 Single Span PSG Bridge, LRFD Design, Rating Example - LRFR

The bridge was built in 2007 and has no deterioration. There is no overlay on the structure.

This example will perform the LRFR rating calculations for the bridge that was designed in Chapter 19 of this manual (E19-1). Though it is necessary to rate both interior and exterior girders to determine the minimum capacity, the below rating will analyze the interior girder only.

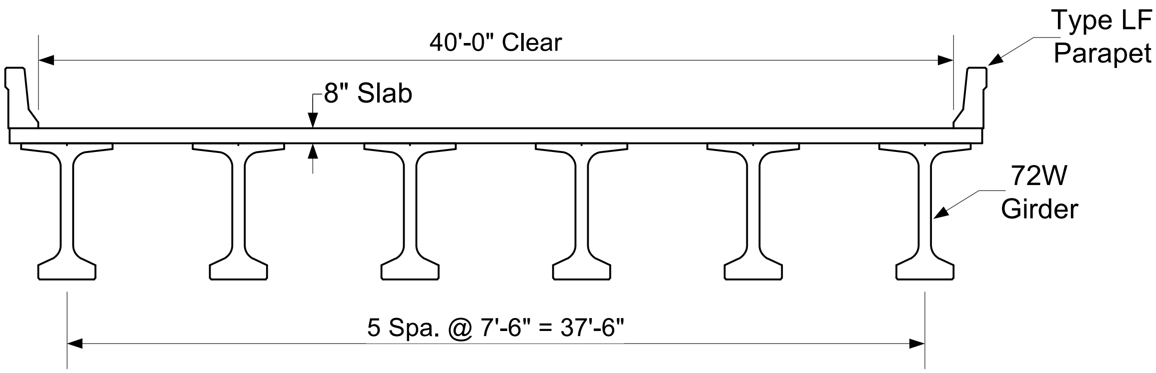


Figure E45-2.1

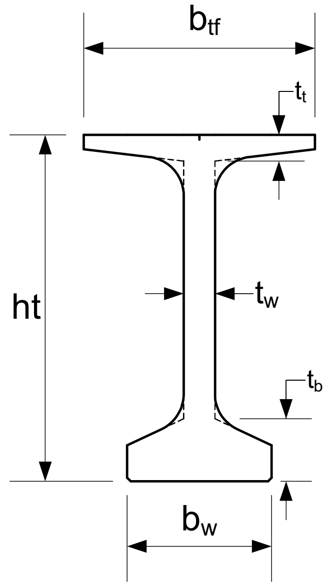
E45-2.1 Preliminary Data

$L := 146$	center to center of bearing, ft
$f'_c := 8$	girder concrete strength, ksi
$f'_{cd} := 4$	deck concrete strength, ksi
$f_{pu} := 270$	strength of low relaxation strand, ksi
$d_b := 0.6$	strand diameter, inches
$A_s := 0.217$	area of strand, in2
$t_s := 8$	slab thickness, in
$t_{se} := 7.5$	effective slab thickness (slab thickness - 1/2 in wearing surface), in
$w_p := 0.387$	weight of Wisconsin Type LF parapet, klf
$w_c := 0.150$	weight of concrete, kcf
$H_{avg} := 2$	average thickness of haunch, in
$w := 40$	clear width of deck, 2 lane road, 3 design lanes, ft
$S := 7.5$	spacing of the girders, ft
$ng := 6$	number of girders

E45-2.2 Girder Section Properties

72W Girder Properties (46 strands, 8 draped):

$b_{tf} := 48$	width of top flange, in
$t_t := 5.5$	avg. thickness of top flange, in
$t_w := 6.5$	thickness of web, in
$t_b := 13$	avg. thickness of bottom flange, in
$ht := 72$	height of girder, in
$b_w := 30$	width of bottom flange, in
$A_g := 915$	area of girder, in²
$I_g := 656426$	moment of inertia of girder, in⁴
$y_t := 37.13$	centriod to top fiber, in
$y_b := -34.87$	centroid to bottom fiber, in
$S_t := 17680$	section modulus for top, in³
$S_b := -18825$	section modulus for bottom, in³
$w_g := 0.953$	weight of girder, klf
$ns := 46$	number of strands
$e_s := -30.52$	centriod to cg strand pattern



$e_g := y_t + 2 + \frac{t_{se}}{2}$ $\boxed{e_g = 42.88}$ in

Web Depth: $d_w := ht - t_t - t_b$ $\boxed{d_w = 53.50}$ in

$E_{beam8} := 5500 \cdot \frac{\sqrt{f'_c \cdot 1000}}{\sqrt{6000}}$ $\boxed{E_{beam8} = 6351}$ $E_B := E_{beam8}$

Modulus of elasticity at time of release (used to for loss calculations):

$E_{beam6.8} := 33000 \cdot K_1 \cdot w_c^{1.5} \cdot \sqrt{f'_{ci}}$ $\boxed{E_{beam6.8} = 4999}$ $E_{ct} := E_{beam6.8}$

$E_D := E_{deck4}$

$n := \frac{E_B}{E_D}$ $\boxed{n = 1.540}$

$K_g := n \cdot \left(I_g + A_g \cdot e_g^{2}\right)$ **LRFD [Eq 4.6.2.2.1-1]** $\boxed{K_g = 3600866}$ in⁴

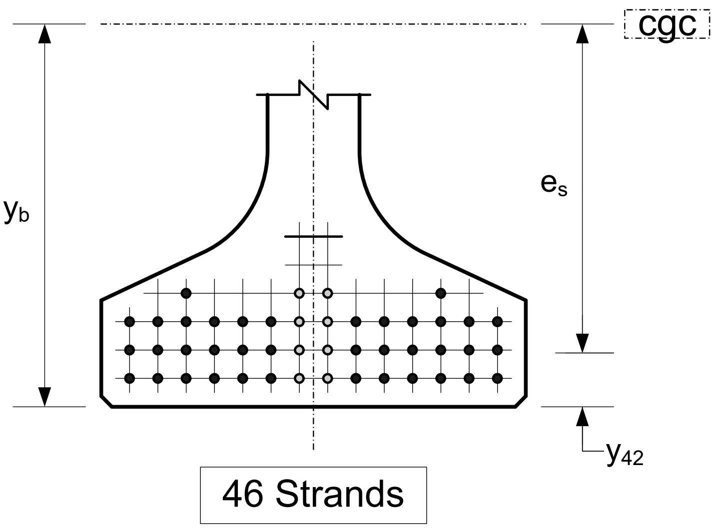


Figure E45-2.2

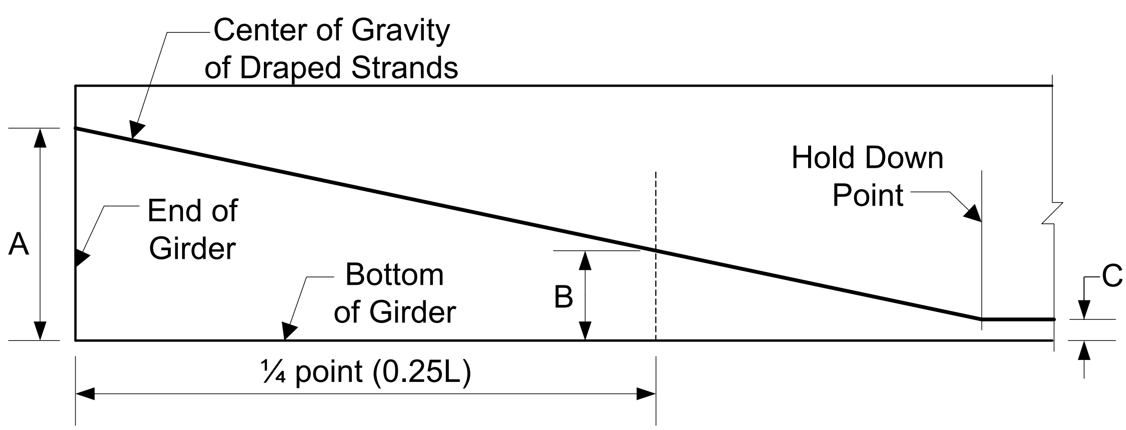


Figure E45-2.3

$A := 67$ in

$C := 5$ in

$B_{min} := 20.5$ in

$B_{max} := 23.5$ in

$$B_{avg} := \frac{B_{min} + B_{max}}{2}$$ $\boxed{B_{avg} = 22.0}$ in

$$slope := \left[\frac{A - B_{avg}}{(0.25) \cdot L \cdot 12}\right] \cdot 100$$ $\boxed{slope = 10.274}$ %

E45-2.3 Composite Girder Section Properties

Calculate the effective flange width in accordance with 17.2.11 and **LRFD [4.6.2.6]**:

$b_{eff} := S \cdot 12$ $\boxed{b_{eff} = 90.00}$ in

The effective width, b_{eff}, must be adjusted by the modular ratio, n, to convert to the same concrete material (modulus) as the girder.

$b_{eadj} := \frac{b_{eff}}{n}$ $\boxed{b_{eadj} = 58.46}$ in

Calculate the composite girder section properties:

effective slab thickness; $\boxed{t_{se} = 7.50}$ in

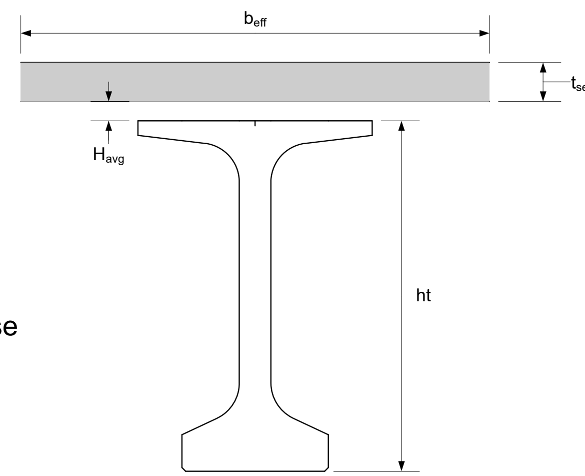
effective slab width; $\boxed{b_{eadj} = 58.46}$ in

haunch thickness; $\boxed{H_{avg} = 2.00}$ in

total height; $h_c := ht + H_{avg} + t_{se}$

$\boxed{h_c = 81.50}$ in

$\boxed{n = 1.540}$



Note: The area of the concrete haunch is not included in the calculation of the composite section properties.

Component	Ycg	A	AY	AY^2	I	I+AY^2
Deck	77.75	438	34089	2650458	2055	2652513
Girder	34.87	915	31906	1112564	656426	1768990
Haunch	73	0	0	0	0	0
Summation		1353	65996			4421503

$\Sigma A := 1353$ in^2

$\Sigma AY := 65996$ in^3

$\Sigma IplusAYsq := 4421503$ in^4

$$y_{cgb} := \frac{-\Sigma AY}{\Sigma A}$$ $\boxed{y_{cgb} = -48.8}$ in

$$y_{cgt} := ht + y_{cgb}$$ $\boxed{y_{cgt} = 23.2}$ in

$$A_{cg} := \Sigma A$$ $\boxed{A_{cg} = 1353}$ in^2

$$I_{cg} := \Sigma IplusAYsq - A_{cg} \cdot y_{cgb}^{\ 2}$$ $\boxed{I_{cg} = 1202381}$ in^4

$$S_{cgt} := \frac{I_{cg}}{y_{cgt}}$$ $\boxed{S_{cgt} = 51777}$ in^3

$$S_{cgb} := \frac{I_{cg}}{y_{cgb}}$$ $\boxed{S_{cgb} = -24650}$ in^3

E45-2.4 Dead Load Analysis - Interior Girder

Dead load on non-composite (DC_1):

weight of 72W girders $\boxed{w_g = 0.953}$ klf

weight of 2-in haunch

$$w_h := \left(\frac{H_{avg}}{12}\right) \cdot \left(\frac{b_{tf}}{12}\right) \cdot (w_c)$$ $\boxed{w_h = 0.100}$ klf

weight of diaphragms $\boxed{w_D := 0.006}$ klf

weight of slab

$$w_d := \left(\frac{t_s}{12}\right) \cdot (S) \cdot (w_c)$$ $\boxed{w_d = 0.750}$ ksf

$$DC_1 := w_g + w_h + w_D + w_d$$ $\boxed{DC_1 = 1.809}$ klf

$$V_{DC1} := \frac{DC_1 \cdot L}{2}$$ $\boxed{V_{DC1} = 132}$ kips

$$M_{DC1} := \frac{DC_1 \cdot L^2}{8}$$ $\boxed{M_{DC1} = 4820}$ kip-ft

* Dead load on composite (DC_2):

weight of single parapet, klf $\boxed{w_p = 0.387}$ klf

weight of 2 parapets, divided equally to all girders, klf

$$DC_2 := \frac{w_p \cdot 2}{ng} \qquad \boxed{DC_2 = 0.129} \text{ klf}$$

$$V_{DC2} := \frac{DC_2 \cdot L}{2} \qquad \boxed{V_{DC2} = 9} \text{ kips}$$

$$M_{DC2} := \frac{DC_2 \cdot L^2}{8} \qquad \boxed{M_{DC2} = 344} \text{ kip-ft}$$

* Wearing Surface (DW): There is no current wearing surface on this bridge. However, it is designed for a 20 psf future wearing surface. Thus, it will be used in the calculations for the Wisconsin Standard Permit Vehicle Design Check, Section 45.12.

$$DW := \frac{w \cdot 0.020}{ng} \qquad \boxed{DW = 0.133} \text{ klf}$$

$$V_{DW} := \frac{DW \cdot L}{2} \qquad \boxed{V_{DW} = 10} \text{ kips}$$

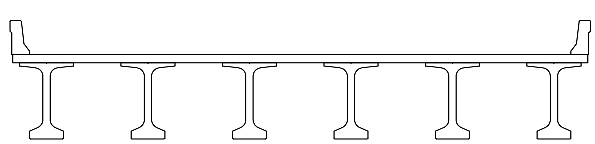
$$M_{DW} := \frac{DW \cdot L^2}{8} \qquad \boxed{M_{DW} = 355} \text{ kip-ft}$$

* **LRFD [4.6.2.2.1]** states that permanent loads on the deck may be distributed uniformly among the beams. This method is used for the parapet and future wearing surface loads.

E45-2.5 Live Load Analysis - Interior Girder

Live Load Distribution Factors (g)

In accordance with **LRFD [Table 4.6.2.2.1-1]**, this structure is a Type "K" bridge.



Distribution factors are in accordance with **LRFD [Table 4.6.2.2.2b-1]**. For an interior beam, the distribution factors are shown below:

For one Design Lane Loaded:

$$0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1}$$

For Two or More Design Lanes Loaded:

$$0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1}$$

E45-2.5.1 Moment Distribution Factors for Interior Beams:

One Lane Loaded:

$$g_{i1} := 0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1} \qquad \boxed{g_{i1} = 0.435}$$

Two or More Lanes Loaded:

$$g_{i2} := 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1} \qquad \boxed{g_{i2} = 0.636}$$

$$g_i := \max(g_{i1}, g_{i2}) \qquad \boxed{g_i = 0.636}$$

Note: The distribution factors above already have a multiple presence factor included that is used for service and strength limit states. For permit load analysis utilizing single lane distribution, the 1.2 multiple presence factor should be divided out.

E45-2.5.2 Shear Distribution Factors for Interior Beams:

One Lane Loaded:

$$g_{v1} := 0.36 + \frac{S}{25} \qquad \boxed{g_{v1} = 0.660}$$

Two or More Lanes Loaded:

$$g_{v2} := 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2 \qquad \boxed{g_{v2} = 0.779}$$

$$g_v := \max(g_{v1}, g_{v2}) \qquad \boxed{g_v = 0.779}$$

E45-2.5.3 Live Load Moments

The unfactored live load load moments (per lane including impact) are listed below (values are in kip-ft). Note that the dynamic load allowance is applied only to the truck portion of the HL-93 loads.

Unfactored Live Load + Impact Moments per Lane (kip-ft)		
Tenth Point	Truck	Tandem
0	0	0
0.1	1783	1474
0.2	2710	2618
0.3	4100	3431
0.4	4665	3914
0.5	4828	4066

The unfactored live load moments per lane are calculated by applying the appropriate distribution factor to the controlling moment. For the interior girder:

$g_i = 0.636$

$M_{LLIM} := g_i \cdot 4828$ $\boxed{M_{LLIM} = 3073}$ kip-ft

E45-2.6 Compute Nominal Flexural Resistance at Midspan

At failure, we can assume that the tendon stress is:

$$f_{ps} = f_{pu}\left(1 - k\cdot\frac{c}{d_p}\right)$$

where:

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right)$$

From **LRFD Table [C5.7.3.1.1-1]**, for low relaxation strands, $k := 0.28$.

"c" is defined as the distance between the neutral axis and the compression face (inches).

Assumed dimensions:

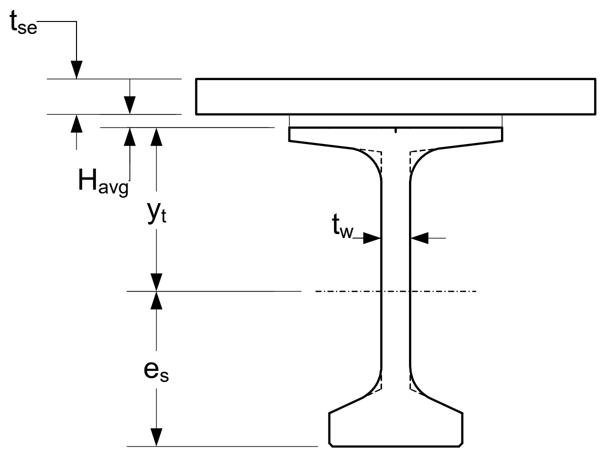


Figure E45-2.4

Assume that the compression block is in the deck. Calculate the capacity as if it is a rectangular section (with the compression block in the flange). The neutral axis location,calculated in accordance with **LRFD 5.7.3.1.1** for a rectangular section, is:

$$c = \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$

where:

$A_{ps} := ns \cdot A_s$ $\boxed{A_{ps} = 9.98}$ in^2

$b := b_{eff}$ $\boxed{b = 90.00}$ in

LRFD [5.7.2.2] $\alpha_1 := 0.85$ (for $f'_{cd} \leq 10.0$ ksi)

$\beta_1 := \max[0.85 - (f'_{cd} - 4) \cdot 0.05, 0.65]$ $\boxed{\beta_1 = 0.850}$

$d_p := y_t + H_{avg} + t_{se} - e_s$ $\boxed{d_p = 77.15}$ in

$$c := \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c = 9.99}$ in

$a := \beta_1 \cdot c$ $\boxed{a = 8.49}$ in

The calculated value of "a" is greater than the deck thickness. Therefore, the rectangular assumption is incorrect and the compression block extends into the haunch. Calculate the neutral axis location and capacity for a flanged section:

$h_f := t_{se}$ depth of compression flange $\boxed{t_{se} = 7.500}$ in

$b_{tf} = 48.00$ width of top flange, inches

$$c := \frac{A_{ps} \cdot f_{pu} - \alpha_1 \cdot f'_{cd} \cdot (b - b_{tf}) \cdot h_f}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b_{tf} + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c = 10.937}$ in

$a := \beta_1 \cdot c$ $\boxed{a = 9.30}$ in

This is above the base of the haunch (9.5 inches) and nearly to the web of the girder. Assume OK.
Now calculate the effective tendon stress at ultimate:

$$f_{ps} := f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right)$$ $\boxed{f_{ps} = 259.283}$ ksi

$T_u := f_{ps} \cdot A_{ps}$ $\boxed{T_u = 2588}$ kips

Calculate the nominal moment capacity of the composite section in accordance with **LRFD [5.7.3.2], [5.7.3.2.2]**:

$$M_n := \left[A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) + \alpha_1 \cdot f'_{cd} \cdot (b - b_{tf}) \cdot h_f \cdot \left(\frac{a}{2} - \frac{h_f}{2}\right)\right] \cdot \frac{1}{12}$$

$\boxed{M_n = 15717}$ kip-ft

For prestressed concrete, $\phi_f := 1.00$, **LRFD [5.5.4.2.1]**. Therefore the usable capacity is:

$M_r := \phi_f \cdot M_n$ $\boxed{M_r = 15717}$ kip-ft

Check Minimum Reinforcement
The amount of reinforcement must be sufficient to develop M_r equal to the lesser of M_{cr} or 1.33 M_u per **LRFD [5.7.3.3.2]**

$\gamma_{LL} := 1.75$ $\gamma_{DC} = 1.250$ $\eta := 1.0$

$Mu := \eta \cdot [\gamma_{DC} \cdot (M_{DC1} + M_{DC2}) + \gamma_{LL} \cdot M_{LLIM}]$ $\boxed{Mu = 11832}$ kip-ft

$\boxed{1.33 \cdot Mu = 15737}$ kip-ft

Calculate M_{cr} next and compare its value with 1.33 Mu

M_{cr} is calculated as follows:

$f_r = 0.24 \cdot \lambda \sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r := 0.24 \cdot \sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.679}$ ksi

$f_{cpe} := \frac{T}{A_g} + \frac{T \cdot e_s}{S_b}$ $\boxed{f_{cpe} = 4.341}$ ksi

$M_{dnc} := M_{DC1}$ $\boxed{M_{dnc} = 4820}$ kip-ft

$S_c := -S_{cgb}$ $\boxed{S_c = 24650}$ ksi

$S_{nc} := -S_b$ $\boxed{S_{nc} = 18825}$ ksi

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_2 := 1.1$ prestress variability factor

$\gamma_3 := 1.0$ for prestressed concrete members

$M_{cr} := \gamma_3 \cdot \left[S_c \cdot \left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe} \right) \cdot \frac{1}{12} - M_{dnc} \cdot \left(\frac{S_c}{S_{nc}} - 1 \right) \right]$ $\boxed{M_{cr} = 10547}$ kip-ft

$M_{cr} = 10547$ kip-ft < $1.33Mu = 15737$, therefore M_{cr} controls

This satisfies the minimum reinforcement check since $M_{cr} < M_r$

Elastic Shortening Loss

at transfer (before ES loss) **LRFD [5.9.5.2]**

$T_{oi} := ns \cdot f_{tr} \cdot A_s$ $\boxed{= 46 \cdot 202.5 \cdot 0.217 = 2021}$ kips

The ES loss estimated above was: $\Delta f_{pES\_est} := 17$ ksi, or $ES_{loss} = 7.900$ %. The resulting force in the strands after ES loss:

$T_o := \left(1 - \frac{ES_{loss}}{100} \right) \cdot T_{oi}$ $\boxed{T_o = 1862}$ kips

If we assume all strands are straight we can calculate the initial elastic shortening loss;

$$f_{cgp} := \frac{T_o}{A_g} + (T_o \cdot e_s) \cdot \frac{e_s}{I_g} + M_g \cdot 12 \cdot \frac{e_s}{I_g}$$ $\boxed{f_{cgp} = 3.240}$ ksi

$\boxed{E_{ct} = 4999}$ ksi

$$E_p := E_s$$ $\boxed{E_p = 28500}$ ksi

$$\Delta f_{pES} := \frac{E_p}{E_{ct}} \cdot f_{cgp}$$ $\boxed{\Delta f_{pES} = 18.471}$ ksi

$$f_i := f_{tr} - \Delta f_{pES}$$ $\boxed{f_i = 184.029}$ ksi

Approximate Estimate of Time Dependant Losses

Calculate the components of the time dependant losses; shrinkage, creep and relaxation, using the approximate method in accordance with **LRFD [5.9.5.3]**.

$$\Delta f_{pLT} = 10.0 \cdot \frac{f_{pi} \cdot A_s}{A_g} \cdot \gamma_h \cdot \gamma_{st} + 12.0 \cdot \gamma_h \cdot \gamma_{st} + \Delta f_{pR}$$

From **LRFD [Figure 5.4.2.3.3-1]**, the average annual ambient relative humidity, $H := 72$ %.

$$\gamma_h := 1.7 - 0.01 \cdot H$$ $\boxed{\gamma_h = 0.980}$

$$\gamma_{st} := \frac{5}{1 + f'_{ci}}$$ $\boxed{\gamma_{st} = 0.641}$

$\Delta f_{pR} := 2.4$ ksi for low relaxation strands

$$\Delta f_{pCR} := 10.0 \cdot \frac{f_{tr} \cdot A_s \cdot ns}{A_g} \cdot \gamma_h \cdot \gamma_{st}$$ $\boxed{\Delta f_{pCR} = 13.878}$ ksi

$$\Delta f_{pSR} := 12.0 \cdot \gamma_h \cdot \gamma_{st}$$ $\boxed{\Delta f_{pSR} = 7.538}$ ksi

$$\Delta f_{pRE} := \Delta f_{pR}$$ $\boxed{\Delta f_{pRE} = 2.400}$ ksi

$$\Delta f_{pLT} := \Delta f_{pCR} + \Delta f_{pSR} + \Delta f_{pRE}$$ $\boxed{\Delta f_{pLT} = 23.816}$ ksi

The total estimated prestress loss (Approximate Method):

$\Delta f_p := \Delta f_{pES} + \Delta f_{pLT}$ $\boxed{\Delta f_p = 42.288}$ ksi

$\boxed{\frac{\Delta f_p}{f_{tr}} \cdot 100 = 20.883}$ % total prestress loss

The remaining stress in the strands and total force in the beam after all losses is:

$f_{pe} := f_{tr} - \Delta f_p$ $\boxed{f_{pe} = 160.21}$ ksi

E45-2.7 Compute Nominal Shear Resistance at First Critical Section

Note: **MBE [6A.5.8]** does not require a shear evaluation for the Design Load Rating or the Legal Load Rating provided the bridge shows no visible sign of shear distress. However, for this example, we will show one iteration for the Design Load Rating.

The shear analysis is always required for Permit Load Rating.

The following will illustrate the calculation at the first critical section only. Due to the variation of resistances for shear along the length of the prestressed concrete I-beam, it is not certain what location will govern. Therefore, a systematic evaluation of the shear and the longitudinal yield criteria based on shear-moment interation should be performed along the length of the beam.

Simplified Procedure for Prestressed and Nonprestressed Sections, **LRFD [5.8.3.4.3]**

$b_v := t_w$ $\boxed{b_v = 6.50}$ in

The critical section for shear is taken at a distance of d_v from the face of the support, **LRFD [5.8.3.2]**.

d_v = effective shear depth taken as the distance between the resultants of the tensile and compressive forces due to flexure. It need not be taken less than the greater of 0.9*d_e or 0.72h (inches). **LRFD [5.8.2.9]**

The first estimate of d_v is calculated as follows:

$d_v := -e_s + y_t + H_{avg} + t_{se} - \frac{a}{2}$ $\boxed{d_v = 72.50}$ in

However, since there are draped strands for a distance of $HD := 49$ from the end of the girder, a revised value of e_s should be calculated based on the estimated location of the critical section. Since the draped strands will raise the center of gravity of the strand group near the girder end, try a smaller value of "d_v" and recalculate "e_s" and "a".

Try $d_v := 65$ inches.

For the standard bearing pad of width, $w_{brg} := 8$ inches, the distance from the end of the girder to the critical section:

$$L_{crit} := \left(\frac{w_{brg}}{2} + d_v\right)\cdot\frac{1}{12} + 0.5 \qquad \boxed{L_{crit} = 6.25} \text{ ft}$$

Calculate the eccentricity of the strand group at the critical section.

$slope = 10.274$

$y_{8t} := A + y_b$

$y_{8t} = 32.130$

$ns_{sb} := 38$ number of undraped strands

$ns_d := 8$ number of draped strands

Find the center of gravity for the 38 straight strands from the bottom of the girder:

$$Y_S := \frac{12\cdot 2 + 12\cdot 4 + 12\cdot 6 + 2\cdot 8}{ns_{sb}} \qquad \boxed{Y_S = 4.211} \text{ in}$$

$$y_s := y_b + Y_S \qquad y_s = -30.659 \text{ in}$$

$$y_{8t\_crit} := y_{8t} - \frac{slope}{100}\cdot L_{crit}\cdot 12 \qquad \boxed{y_{8t\_crit} = 24.42} \text{ in}$$

$$e_{s\_crit} := \frac{ns_{sb}\cdot y_s + ns_d\cdot y_{8t\_crit}}{ns_{sb} + ns_d} \qquad \boxed{e_{s\_crit} = -21.08} \text{ in}$$

Calculation of compression stress block based on revised eccentricity:

$$d_{p\_crit} := y_t + H_{avg} + t_{se} - e_{s\_crit} \qquad \boxed{d_{p\_crit} = 67.71} \text{ in}$$

Note that the area of steel is based on the number of bonded strands.

$$A_{ps\_crit} := (ns)\cdot A_s \qquad \boxed{A_{ps\_crit} = 9.98} \text{ in}^2$$

Also, the value of f_{pu}, should be revised if the critical section is located less than the development length from the end of the beam. The development length for a prestressing strand is calculated in accordance with **LRFD [5.11.4.2]**:

$K := 1.6$ for prestressed members with a depth greater than 24 inches

$\boxed{d_b = 0.600}$ in

$$l_d := K \cdot \left(f_{ps} - \frac{2}{3} \cdot f_{pe}\right) \cdot d_b \qquad \boxed{l_d = 146.4} \text{ in}$$

The transfer length may be taken as: $l_{tr} := 60 \cdot d_b$ $\boxed{l_{tr} = 36.00}$ in

Since $L_{crit} = 6.250$ feet is between the transfer length and the development length, the design stress in the prestressing strand is calculated as follows:

$$f_{pu\_crit} := f_{pe} + \frac{L_{crit} \cdot 12 - l_{tr}}{l_d - l_{tr}} \cdot \left(f_{ps} - f_{pe}\right) \qquad \boxed{f_{pu\_crit} = 195} \text{ ksi}$$

For rectangular section behavior:

$$c := \frac{A_{ps\_crit} \cdot f_{pu\_crit}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps\_crit} \cdot \dfrac{f_{pu\_crit}}{d_{p\_crit}}} \qquad \boxed{c = 7.267} \text{ in}$$

$$a_{crit} := \beta_1 \cdot c \qquad \boxed{a_{crit} = 6.177} \text{ in}$$

Calculation of shear depth based on refined calculations of e_s and a:

$$d_{v\_crit} := -e_{s\_crit} + y_t + H_{avg} + t_{se} - \frac{a_{crit}}{2} \qquad \boxed{d_{v\_crit} = 64.62} \text{ in}$$

This value matches the assumed value of d_v above. OK!

The nominal shear resistance of the section is calculated as follows, **LRFD [5.8.3.3]**:

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

where $V_p := 0$ in the calculation of V_n, if the simplified procedure is used (**LRFD [5.8.3.4.3]**).

V_d = shear force at section due to unfactored dead load and includes both DC and DW (kips)

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max} (kips). (Not necessarily equal to V_u.)

M_{cre} = moment causing flexural cracking at section due to externally applied loads (kip-in)

M_{max} = maximum factored moment at section due to externally applied loads (kip-in)

M_{dnc} = total unfactored dead load moment acting on the noncomposite section (kip-ft)

Values for the following moments and shears are at the critical section, $L_{crit} = 6.25$ feet from the end of the girder at the abutment.

$\boxed{V_{DCnc} = 121.7}$ kips

$\boxed{V_{DCc} = 8.7}$ kips

$\boxed{V_{DWc} = 9.0}$ kips

$Vi_{LL} := Vi_{LL\_lane} \cdot g_{vi}$ $\boxed{Vi_{LL} = 100.5}$ kips

$V_i := 1.75 \cdot Vi_{LL}$ $\boxed{V_i = 175.9}$ kips

$V_d := V_{DCc} + V_{DCnc} + V_{DWc}$ $\boxed{V_d = 139.3}$ kips

$V_u := 1.25 \cdot (V_{DCnc} + V_{DCc}) + 1.5 \cdot V_{DWc} + 1.75 \cdot Vi_{LL}$ $\boxed{V_u = 352.2}$ kips

$M_{dnc} := 730$ kip-ft

$M_{max} := 837$ kip-ft

However, the equations below require the value of M_{max} to be in kip-in:

$\boxed{M_{max} = 10044}$ kip-in

$f_r = -0.20 \cdot \lambda \sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r := -0.20 \cdot \sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = -0.566}$ ksi

$T_{crit} := A_{ps\_crit} \cdot f_{pe}$ $\boxed{T_{crit} = 1599}$ kips

$f_{cpe} := \frac{T_{crit}}{A_g} + \frac{T_{crit} \cdot e_{s\_crit}}{S_b}$ $\boxed{f_{cpe} = 3.539}$ ksi

$\boxed{M_{dnc} = 730}$ kip-ft

$\boxed{M_{max} = 10044}$ kip-in

$S_c := S_{cgb}$ $\boxed{S_c = -24650}$ in^3

$S_{nc} := S_b$ $\boxed{S_{nc} = -18825}$ in^3

$M_{cre} := S_c \cdot \left(f_r - f_{cpe} - \frac{12 M_{dnc}}{S_{nc}}\right)$ $\boxed{M_{cre} = 89699}$ kip-in

Calculate V_{ci}, **LRFD [5.8.3.4.3]** λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$V_{ci1} := 0.06 \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v$ $\boxed{V_{ci1} = 71.7}$ kips

$V_{ci2} := 0.02 \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v + V_d + \frac{V_i \cdot M_{cre}}{M_{max}}$ $\boxed{V_{ci2} = 1733.9}$ kips

$V_{ci} := \max(V_{ci1}, V_{ci2})$ $\boxed{V_{ci} = 1733.9}$ kips

$f_t := \frac{T_{crit}}{A_g} + \frac{T_{crit} \cdot e_{s\_crit}}{S_t} + \frac{M_{dnc} \cdot 12}{S_t}$ $\boxed{f_t = 0.337}$ ksi

$f_b := \frac{T_{crit}}{A_g} + \frac{T_{crit} \cdot e_{s\_crit}}{S_b} + \frac{M_{dnc} \cdot 12}{S_b}$ $\boxed{f_b = 3.073}$ ksi

$\boxed{y_{cgb} = -48.78}$ in

$\boxed{ht = 72.00}$ in

$f_{pc} := f_b - y_{cgb} \cdot \frac{f_t - f_b}{ht}$ $\boxed{f_{pc} = 1.219}$ ksi

$V_{p\_cw} := ns_d \cdot A_s \cdot f_{pe} \cdot \frac{slope}{100}$ $\boxed{V_{p\_cw} = 28.6}$ kips

Calculate V_{cw}, **LRFD [5.8.3.4.3]** λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$V_{cw} := \left(0.06 \cdot \lambda \sqrt{f'_c} + 0.30 \cdot f_{pc}\right) \cdot b_v \cdot d_v + V_{p\_cw}$ $\boxed{V_{cw} = 254.8}$ kips

$V_c := \min(V_{ci}, V_{cw})$ $\boxed{V_c = 254.8}$ kips

Calculate the shear resistance at L_{crit}:

$\phi_v := 0.9$ **LRFD [5.5.4.2]**

$\boxed{s := 20}$ in

$A_v := 0.40$ in² for #4 rebar

$f_y := 60$ ksi

$\boxed{d_v = 65.00}$ in

$$\cot\theta := \begin{cases} 1 & \text{if } V_{ci} < V_{cw} \\ \min\left(1.0 + 3\cdot\frac{f_{pc}}{\sqrt{f'_c}}, 1.8\right) & \text{otherwise} \end{cases}$$

$\boxed{\cot\theta = 1.800}$

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

LRFD Eq 5.8.3.3-4 reduced per **C5.8.3.3-1** when α = 90 degrees.

$\boxed{V_s = 140}$ kips

$Vn_1 := V_c + V_s + V_p$ $\boxed{Vn_1 = 395}$ kips

$Vn_2 := 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p$ $\boxed{Vn_2 = 845}$ kips

$V_n := \min(Vn_1, Vn_2)$ $\boxed{V_n = 395}$ kips

$V_r := \phi_v \cdot V_n$ $\boxed{V_r = 355.69}$ kips

E45-2.8 Longitudinal Tension Flange Capacity:

The total capacity of the tension reinforcing must meet the requirements of **LRFD [5.8.3.5].** The capacity is checked at the critical section for shear:

$$V_u := 1.25 \cdot (V_{DC1} + V_{DC2}) + 1.50 \cdot (V_{DW}) + 1.75 \cdot (Vu_{LL})$$

$\boxed{V_u = 367.320}$ kips

$$T_{ps} := \frac{M_{max}}{d_v \cdot \phi_f} + \left(\frac{V_u}{\phi_v} - 0.5 \cdot V_s - V_{p\_cw}\right) \cdot \cot\theta$$

$\boxed{T_{ps} = 711}$ kips

actual capacity of the straight bonded strands:

$\boxed{ns_{sb} \cdot A_s \cdot f_{pu\_crit} = 1610}$ kips

Is the capacity of the straight bonded strands greater than T_{ps}? $\boxed{check = \text{"OK"}}$

Check the tension Capacity at the edge of the bearing:

The strand is anchored $l_{px} := 10$ inches. The transfer and development lengths for a prestressing strand are calculated in accordance with **LRFD [5.11.4.2]**:

$\boxed{l_{tr} = 36.00}$ in

$\boxed{l_d = 146.4}$ in

Since l_{px} is less than the transfer length, the design stress in the prestressing strand is calculated as follows:

The assumed crack plane crosses the centroid of the straight strands at

$l_{px'} := l_{px} + Y_S \cdot \cot\theta$ $\boxed{Y_S = 4.211}$ in $\boxed{l_{px'} = 17.58}$ in

$$f_{pb} := \frac{f_{pe} \cdot l_{px'}}{60 \cdot d_b}$$ $\boxed{f_{pb} = 78.23}$ kips

Tendon capacity of the straight bonded strands: $\boxed{ns_{sb} \cdot A_s \cdot f_{pb} = 645}$ kips

The values of V_u, V_s, V_p and θ may be taken at the location of the critical section.

Over the length d_v, the average spacing of the stirrups is:

$$s_{ave} := \frac{6 \cdot 4.5 + 3 \cdot s}{9}$$ $\boxed{s_{ave} = 9.67}$ in

$$V_s := A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{s_{ave}}$$ $\boxed{V_s = 290}$ kips

The vertical component of the draped strands is: $\boxed{V_{p\_cw} = 29}$ kips

The factored shear force at the critical section is: $\boxed{V_{u\_crit} = 352}$ kips

E45-2.9 Design Load Rating

At the Strength I Limit State:

$$RF = \frac{(\phi_c)(\phi_s)(\phi)R_n - \gamma_{DC}(DC_1) - \gamma_{DW}(DW_1)}{\gamma_L(LL + IM)}$$

Live Load Factors taken from Table 45.3-1

$\gamma_{L\_inv} := 1.75$ $\gamma_{DC} := 1.25$ $\gamma_{servLL} := 0.8$

$\gamma_{L\_op} := 1.35$ $\phi_c := 1.0$ $\phi_s := 1.0$

$\phi := 1.0$ for flexure

$\phi := 0.9$ for shear

For Flexure

Inventory Level

$$RF_{Mom\_Inv} := \frac{(1)(1)(1)(M_n) - \gamma_{DC} \cdot (M_{DC1} + M_{DC2})}{\gamma_{L\_inv} \cdot (M_{LLIM})}$$

$$\boxed{RF_{Mom\_Inv} = 1.723}$$

Operating Level

$$RF_{Mom\_Op} := \frac{(1)(1)(1)(M_n) - \gamma_{DC} \cdot (M_{DC1} + M_{DC2})}{\gamma_{L\_op} \cdot (M_{LLIM})}$$

$$\boxed{RF_{Mom\_Op} = 2.233}$$

For Shear at first critical section

Inventory Level

$$RF_{shear\_Inv} := \frac{(1)(1)(0.9)(V_n) - \gamma_{DC} \cdot (V_{DCnc} + V_{DCc})}{\gamma_{L\_inv} \cdot (Vi_{LL})}$$

$$\boxed{RF_{shear\_Inv} = 1.096}$$

Operating Level

$$RF_{shear\_Op} := \frac{(1)(1)(0.9)(V_n) - \gamma_{DC} \cdot (V_{DCnc} + V_{DCc})}{\gamma_{L\_op} \cdot (Vi_{LL})}$$

$$\boxed{RF_{shear\_Op} = 1.421}$$

At the Service III Limit State (Inventory Level):

$$RF = \frac{f_R - \gamma_D \cdot (f_D)}{\gamma_{servLL}(f_{LLIM})}$$

$T := ns \cdot A_s \cdot f_{pe}$ $\boxed{T = 1599}$ kips

$f_{pb} := \frac{T}{A_g} + \frac{T \cdot (e_s)}{S_b}$ $\boxed{f_{pb} = 4.341}$ ksi

Allowable Tensile Stress **LRFD [5.9.4.2.2]**

$t_{all} = -0.19 \cdot \lambda \sqrt{f'_c}$ $\lambda = 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]**

$t_{all} := -0.19 \cdot \sqrt{f'_c}$; $|\ t_{all}\ | \leq 0.6$ ksi $\boxed{t_{all} = -0.537}$ ksi

$f_R := f_{pb} - t_{all}$ $\boxed{f_R = 4.878}$ ksi

Live Load Stresses:

$f_{LLIM} := \frac{M_{LLIM} \cdot 12}{S_{cgb}}$ $\boxed{f_{LLIM} = 1.496}$ ksi

Dead Load Stresses:

$f_{DL} := \frac{M_{DC1} \cdot 12}{S_b} + \frac{M_{DC2} \cdot 12}{S_{cgb}}$ $\boxed{f_{DL} = 3.240}$ ksi

$RF_{serviceIII} := \frac{f_R - 1.0 \cdot (f_{DL})}{\gamma_{servLL} \cdot (f_{LLIM})}$ $\boxed{RF_{serviceIII} = 1.369}$

E45-2.10 Legal Load Rating

Since the Operating Design Load Rating RF>1.0, the Legal Load Rating is not required. The Legal Load computations that follow have been done for illustrative purposes only. Shear ratings have not been illustrated.

Live Loads used will be the AASHTO Legal Loads per Figure 45.10-1 and AASHTO Specialized Hauling Vehicles per Figure 45.10-2.

$\boxed{g_i = 0.636}$

$IM := 33$ %

* WisDOT does not allow for a dynamic load allowance reduction based on the smoothness of the roadway surface. Thus, IM=33%

At the Strength I Limit State:

$$RF = \frac{(\phi_c)(\phi_s)(\phi)R_n - \gamma_{DC}(DC_1) - \gamma_{DW}(DW_1)}{\gamma_L(LL + IM)}$$

Live Load Factors taken from Tables 45.3-1 and 45.3-2

$\phi_c := 1.0$ $\phi_s := 1.0$

$\phi := 1.0$

$\gamma_{L\_Legal} := 1.45$ $\gamma_{DC} := 1.25$

$\gamma_{L\_SU} := 1.45$

For Flexure

$$RF_{Legal} := \frac{(1)(1)(1)(M_n) - \gamma_{DC} \cdot (M_{DC1} + M_{DC2})}{\gamma_{L\_Legal} \cdot (M_{LLIM})}$$

$$RF_{SU} := \frac{(1)(1)(1)(M_n) - \gamma_{DC} \cdot (M_{DC1} + M_{DC2})}{\gamma_{L\_SU} \cdot (M_{LLIM})}$$

AASHTO Type	Truck Type	Truck Weight (Tons)	M_{LL} (Per Lane) (ft-kips)	M_{LLIM} (M_{LL} * IM * g_i) ft-kips	RF Strength I Flexure	Safe Load Capacity (Tons)	Posting?
Commercial Trucks	Type 3	25	1671.0	1413.4	4.520	113	No
	Type 3S2	36	2150.0	1818.6	3.513	126	No
	Type 3-3	40	2260.0	1911.7	3.342	134	No
Specialized Hauling Vehicles	SU4	27	1831.0	1548.8	4.124	111	No
	SU5	31	2062.8	1744.9	3.661	113	No
	SU6	34.75	2294.6	1940.9	3.291	114	No
	SU7	38.75	2540.8	2149.2	2.972	115	No

As expected, all rating factors are well above 1.0. However, if any of the rating factors would have fallen below 1.0, the posting capacity would have been calculated per 45.10.3.2:

$$\text{Posting} := \left(\frac{W}{0.7}\right)[(RF) - 0.3]$$

E45-2.11 Permit Load Rating

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed per 45.12.

The bridge shall be analyzed for this vehicle considering both single-lane and multi-lane distribution. Also, the vehicle will be analyzed assuming it is mixing with other traffic on the bridge and that full dynamic allowance is utilized. Future wearing surface shall be included.

Since this example is rating a newly designed bridge, an additional check is required. The designer shall ensure that the results of the single-lane analysis are greater than 190 kips MVW.

Also, divide out the 1.2 multiple presence factor per **MBE [6A.4.5.4.2]** for the single lane distribution factor run.

For 146' span:

$M190_{LL} := 4930.88$ kip-ft per lane

$V190_{LL} := 145.08$ kips at $\boxed{d_v = 65}$ in

for Strength Limit State

Single Lane Distribution w/ Future Wearing surface (Design check per 45.12)

$g_{m1} := 0.435 \frac{1}{1.2}$ $\boxed{g_{m1} = 0.363}$

$g_{v1} := .660 \cdot \frac{1}{1.2}$ $\boxed{g_{v1} = 0.550}$

For flexure:

$M_{190LLIM} := M190_{LL} \cdot g_{m1} \cdot 1.33$ $\boxed{M_{190LLIM} = 2377}$ kip-ft

$$RF_{190\_moment} := \frac{[(1)(1)(1)M_n] - 1.25 \cdot (M_{DC1} + M_{DC2}) - 1.5 \cdot (M_{DW})}{1.2(M_{190LLIM})}$$

$\boxed{RF_{190\_moment} = 3.060}$

$Wt := RF_{190\_moment} \cdot 190$ $\boxed{Wt = 581}$ kips >> 190 kips, OK

For shear:

$V_{190LLIM} := V190_{LL} \cdot g_{v1} \cdot 1.33$ $\boxed{V_{190LLIM} = 106}$ kips

$$RF_{190\_shear} := \frac{[(1)(1)(0.9)V_n] - 1.25 \cdot (V_{DCnc} + V_{DCc}) - 1.5(V_{DW})}{1.2(V_{190LLIM})}$$

$\boxed{RF_{190\_shear} = 1.399}$

$Wt := RF_{190\_shear} \cdot 190$ $\boxed{Wt = 266}$ kips > 190 kips, OK

Single Lane Distribution w/o Future Wearing surface (For plans and rating sheet only)

$g_{m1} := 0.435 \frac{1}{1.2}$ $\boxed{g_{m1} = 0.363}$

$g_{v1} := .660 \cdot \frac{1}{1.2}$ $\boxed{g_{v1} = 0.550}$

For flexure:

$M_{190LLIM} := M190_{LL} \cdot g_{m1} \cdot 1.33$ $\boxed{M_{190LLIM} = 2377}$ kip-ft

$$RF_{190\_moment} := \frac{[(1)(1)(1)M_n] - 1.25\cdot(M_{DC1} + M_{DC2})}{1.2(M_{190LLIM})}$$

$\boxed{RF_{190\_moment} = 3.247}$

$Wt := RF_{190\_moment}\cdot 190$ $\boxed{Wt = 617}$

For shear:

$V_{190LLIM} := V190_{LL}\cdot g_{v1}\cdot 1.33$ $\boxed{V_{190LLIM} = 106}$ kips

$$RF_{190\_shear} := \frac{[(1)(1)(0.9)V_n] - 1.25\cdot(V_{DCnc} + V_{DCc})}{1.2(V_{190LLIM})}$$

$\boxed{RF_{190\_shear} = 1.514}$

$Wt := RF_{190\_shear}\cdot 190$ $\boxed{Wt = 288}$

Multi-Lane Distribution w/o Future Wearing Surface (For plans and rating sheet only)

$g_{m2} := 0.636$ $\boxed{g_{m2} = 0.636}$

$g_{v2} := .779$ $\boxed{g_{v2} = 0.779}$

For flexure:

$M_{190LLIM} := M190_{LL}\cdot g_{m2}\cdot 1.33$ $\boxed{M_{190LLIM} = 4171}$ kip-ft

$$RF_{190\_moment} := \frac{[(1)(1)(1)M_n] - 1.25\cdot(M_{DC1} + M_{DC2})}{1.3(M_{190LLIM})}$$

$\boxed{RF_{190\_moment} = 1.708}$

$Wt := RF_{190\_moment}\cdot 190$ $\boxed{Wt = 325}$

For shear:

$V_{190LLIM} := V190_{LL} \cdot g_{v2} \cdot 1.33$ $\boxed{V_{190LLIM} = 150}$ kips

$$RF_{190\_shear} := \frac{(1)(1)(0.9)V_n - 1.25 \cdot (V_{DCnc} + V_{DCc})}{1.3(V_{190LLIM})}$$

$\boxed{RF_{190\_shear} = 0.987}$

$Wt := RF_{190\_shear} \cdot 190$ $\boxed{Wt = 187}$

E45-2.12 Summary of Rating Factors

Interior Girder								
Limit State		Design Load Rating		Legal Load Rating	Permit Load Rating (kips)		
		Inventory	Operating		Single Lane w/ FWS	Single Lane w/o FWS	Multi Lane w/o FWS
Strength I	Flexure	1.723	2.233	N/A	581	617	325
	Shear	1.096	1.421	N/A	266	288	187
Service III		1.369	N/A	N/A	N/A	N/A	N/A
Service I		N/A	N/A	N/A	Optional	Optional	Optional

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