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2.1 Organizational Charts

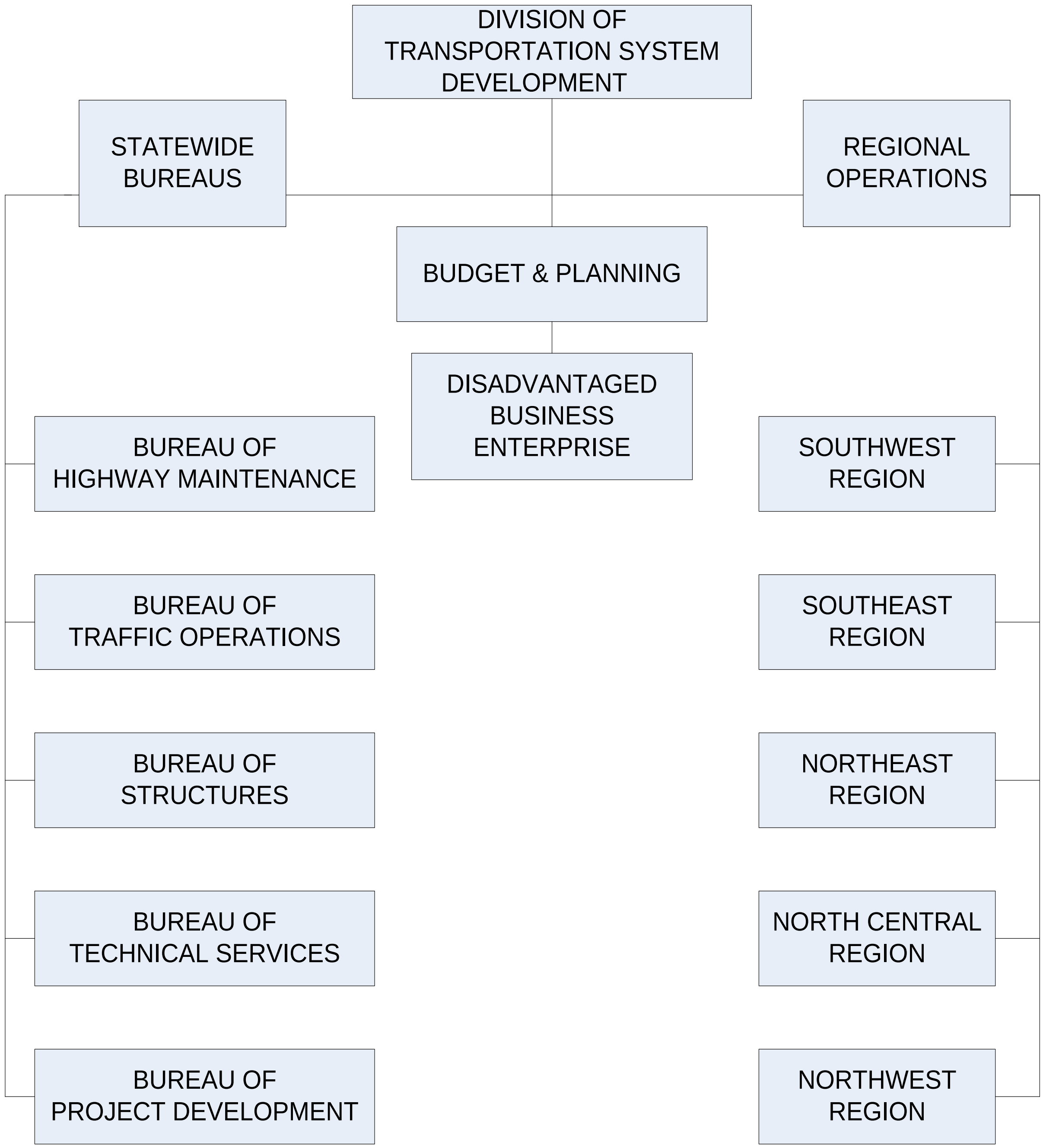

Figure 2.1-1
Division of Transportation System Development

2.5 Structure Numbers

An official number referred to as the Structure Number is assigned to every structure on the State Highway System for the purpose of having a definite designation. The Structure Number is hyphenated with the first letter being either a B, C, P, S, R, M or N. B is assigned to all structures over 20 ft. in length, including culvert configurations. In general, C is assigned to structures 20 ft. or less in length, measured along the skew between the inside faces of the exterior walls, with an exception being that box culverts must have a cross sectional area greater than, or equal to 20 square feet to be assigned a C number. Do not include pipes unless they meet the definition of a bridge. A set of nested pipes may be given a Bridge Number if the distance between the outside walls of the end pipes exceeds 20 ft. and the clear distance between pipe openings is less than half the diameter of the smallest pipe. P designates structures for which there are no structural plans on file. S is for sign structures, R is for Retaining Walls, and N is for Noise Barriers. M is for miscellaneous structures where it is desirable to have a plan record. Bridges on state boundary lines also have a number designated by the adjacent state.

WisDOT Policy Item:

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

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

For guidance on inspection and documentation of state-owned small bridge structures (C-XX-XXX), see the Structures Inspection Manual.

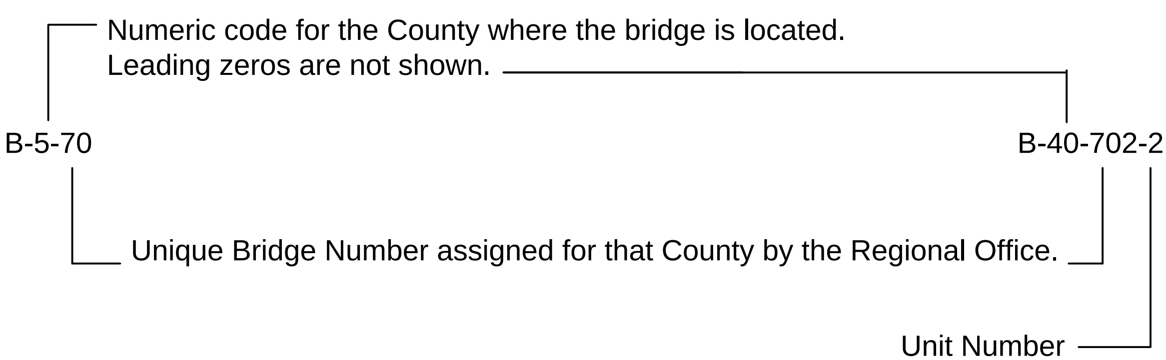

Figure 2.5-1
Bridge Number Detail

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

See Section 6.3.3.7 for guidance on location of name plate on structures.

When a structure is rehabilitated, the name plate should be preserved, if possible, and reinstalled on the rehabilitated Structure. If a new name plate is required, it should show the year of original construction. The original structure number applies to all rehabilitation including widening, lengthening, superstructure replacement, etc.

Pedestrian only bridges get a B number if they are state maintained or cross a roadway. Otherwise use an M number for tracking purposes such as DNR bridges reviewed by DOT.

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3.1 Specifications and Standards

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

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

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

3.2 Geometrics and Loading

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

For highway structures, the minimum desirable longitudinal vertical gradient is 0.5 percent. There have been ponding problems on bridges with smaller gradients. This requirement is applied to the bridge in its final condition, without consideration of short term camber effects. Vertical curves with the high point located on the bridge are acceptable provided that sufficient grade each side of the high point is provided to facilitate drainage. Keeping the apex of the curve off of a pier, especially for slab bridges, can be beneficial to reduce ponding at those locations.

The clearances required on highway crossings are given in the *Facilities Development Manual* (FDM). The recommended clearance for railroad crossings is shown on Chapter 38 Standard for Highway Over Railroad Design Requirements. Proposed railroad clearances are subject to review by the railroad involved.

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

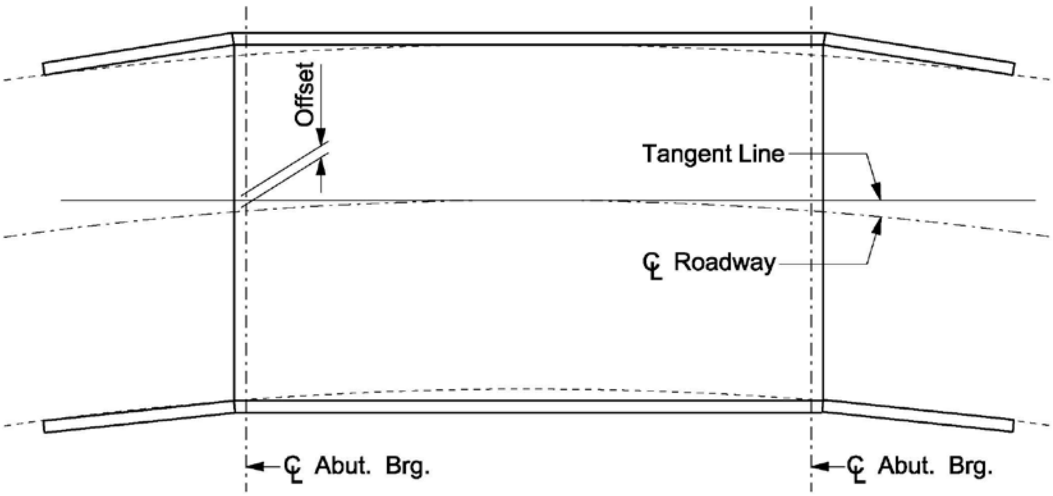
Railroad loadings are specified in the *AREMA Manual for Railway Engineering*.

All new bridges constructed in the State of Wisconsin are designed for the clearances shown in FDM Procedure 11-35-1, Attachment 1.8. FDM Procedure 11-35-1, Attachment 1.9 covers the cases described in that section as well as bridge widenings. Wires and cables over highways are designed for clearances of 18’-0” to 22’-0”. Vertical clearance is needed for the entire roadway width (critical point to include traveled way, auxiliary lanes, turn lanes and shoulders).

Coordinate early in the design process with the Bureau of Highway Maintenance and Bureau of Structures in determining the appropriate vertical clearance along an OSOW High Clearance Route for new bridges, replacement bridges, bridges with superstructure replacement and overhead utilities. Refer to the Facilities Development Manual Chapter 11-10 Section 5.4.3 and Chapter 11-35 Section 1.5.1 for additional details along these high routes, including for new and replacement sign structures.

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

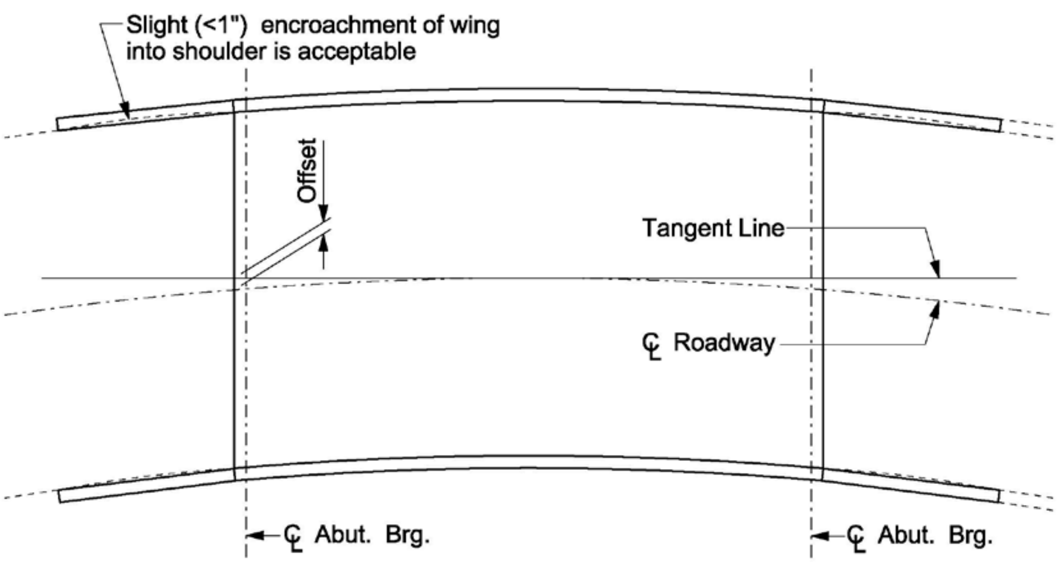
The length of bridge approaches should be determined using appropriate design standards. Refer to FDM 3.5.6 for discussion of touchdown points on local program bridge projects.



Case 1

For offsets 0" to 6"

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



Case 2

For offsets over 6"

Curve entire bridge. Do not widen. Align straight wings so inside of wing tip is at edge of shoulder.

Figure 3.2-1

Bridge Layout on Horizontal Curves

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5.1 Factors Governing Bridge Costs

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

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

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

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

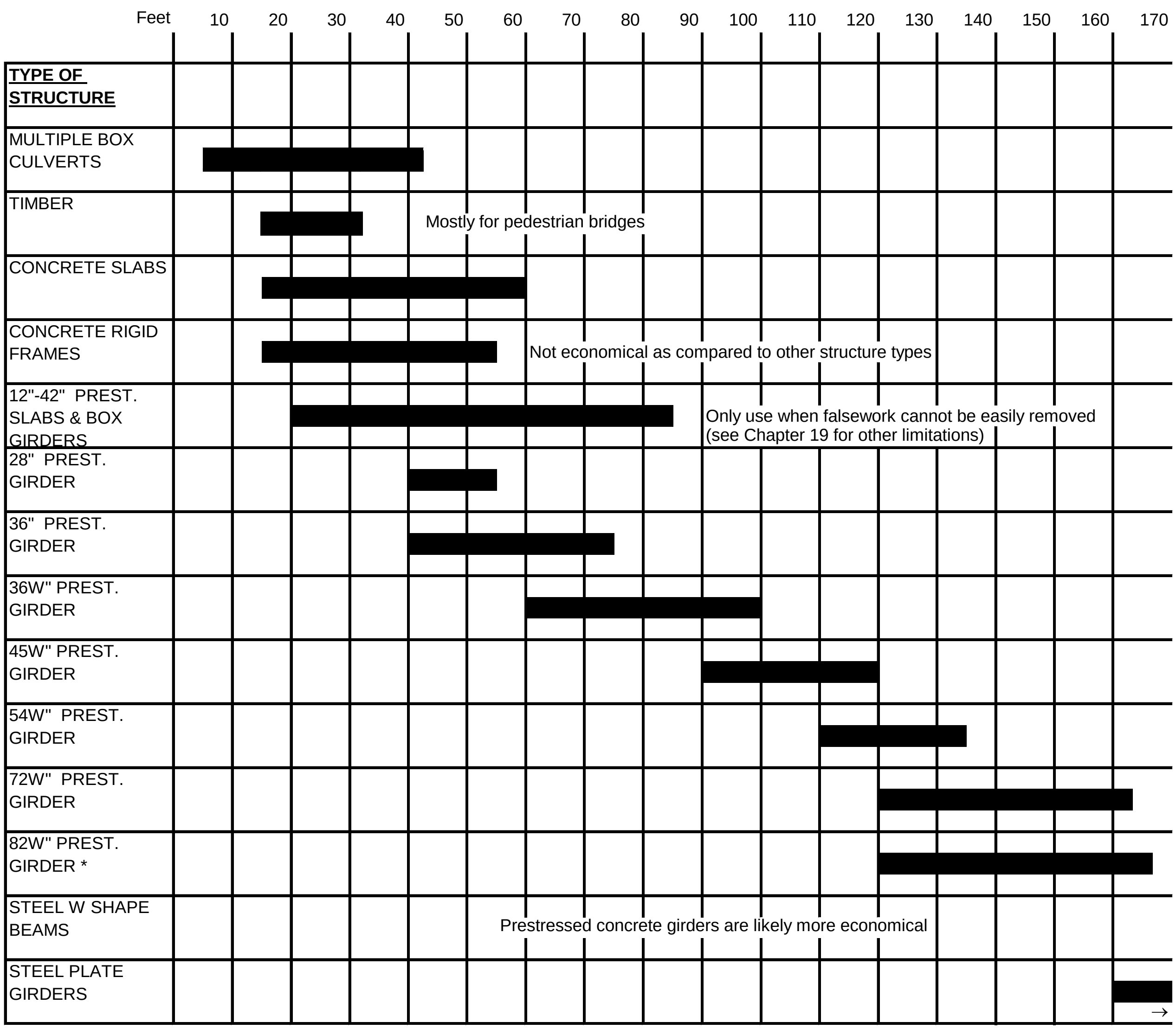
In these reports prestressed girder costs are grouped together because there is a small cost difference between girder sizes. Refer to unit costs. Concrete slab costs are also grouped together for this reason.

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

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

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
Note: Slab bridges should not be used on the Interstate

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	210.97
502.3110.S	Expansion Device Modular (structure) (LS)	LF	969.95
SPV.0105	Expansion Device Modular LRFD (structure) (LS)	LF	1,947.75

Table 5.3-1
Contract Unit Bid Prices for Structures - 2016

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

5.4 Bid Letting Cost Data

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

5.4.1 2012 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	18	115,512	11,610,435	53.88	100.50
Reinf. Conc. Slabs (All but A5)	22	80,797	8,269,942	53.04	102.35
Reinf. Conc. Slabs (A5 Abuts)	3	6,438	739,983	53.24	114.95

Table 5.4-1
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	58	697,381	65,044,526	65.91	93.27
Reinf. Conc. Slabs (All but A5)	1	5,812	491,683	43.73	84.60

Table 5.4-2
Grade Separation Structures

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

Table 5.4-3
Box Culverts

Bridge Type	Cost per Sq. Ft.
Pre-Fab Pedestrian Bridge (B-40-761/762)	325.22
Prestressed Concrete Girder Bridge (B-53-265)	91.93
Buried Slab Bridge (C-13-155)	170.77

Table 5.4-4
Miscellaneous Bridges

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

Table 5.4-5
Retaining Walls

5.4.2 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

5.4.3 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

5.4.4 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

5.4.5 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

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

Point on Curvature	P.O.C.	Standard	STD.
Point on Tangent	P.O.T.	Station	STA.
Property Line	P.L.	Structural	STR.
Quantity	QUAN.	Substructure	SUBST.
Radius	R.	Superstructure	SUPER.
Railroad	R.R.	Surface	SURF.
Railway	RY.	Superelevation	S.E.
Reference	REF.	Symmetrical	SYM
Reinforcement	REINF.	Tangent Line	TAN. LN.
Reinforced Concrete Culvert Pipe	R.C.C.P.	Transit Line	T/L
Required	REQ'D.	Transverse	TRAN.
Right	RT.	Variable	VAR.
Right Hand Forward	R.H.F.	Vertical	VERT.
Right of Way	R/W	Vertical Curve	V.C.
Roadway	RDWY.	Volume	VOL.
Round	ø	West	W.
Section	SEC.	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

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

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

6.3.3.2 Box Culverts

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

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

Bid items commonly used are excavation, concrete masonry, bar steel, rubberized membrane waterproofing, backfill and rip rap. Filler is a non-bid item. In lieu of showing a contour map, show profile grade lines as described for Subsurface Exploration sheet.

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

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

6.3.3.3 Miscellaneous Structures

Detail plans for other structures such as retaining walls, pedestrian bridges, and erosion control structures are to be detailed with the same requirements as previously mentioned. Multiple sign structure of the same type and project may be combined into a single set of plans per standard insert sheet provisions, and shall be subject to the same requirements for bridge plans.

6.3.3.4 Standard Drawings

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

6.3.3.5 Insert Sheets

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

6.3.3.6 Change Orders and Maintenance Work

These plans are drawn on full size sheets. A Structure Survey Report should be submitted for all maintenance projects, including painting projects and polymer overlay projects. In addition to the SSR, final structure plans on standard sheet borders with the #8 tab should be submitted to BOS in the same fashion as other rehabilitation plans. Painting plans should include at minimum a plan view with overall width and length dimensions, the number of spans, an indication of the number and type of elements to be painted (girders, trusses, etc.), and an elevation view showing what the structure is crossing. The SSR should give a square foot quantity for patchwork painting. For entire bridges or well defined zones (e.g. Paint all girders 5 feet on each side of expansion joints), the design engineer will be responsible for determining the quantity.

6.3.3.7 Name Plate and Bench Marks

WisDOT has discontinued the statewide practice of furnishing bench mark disks and requiring them to be placed on structures. However, WisDOT Region Offices may continue to provide bench mark disks for the contract to be set. When requested, bench mark disks shall be shown on bridge and larger culvert plans. Locate the bench mark disks on a horizontal surface flush with the concrete. Bench marks to be located on top of the parapet on the bridge deck, above the first right corner of the abutment traveling in the highway cardinal directions of North or East. See FDM 9-25-5 for additional bench mark information. For multi-directional bridges, locate the name plate on the roadway side of the first right wing or parapet traveling in the highway cardinal directions of North or East. For one-directional bridges, locate the name plate on the first right wing or parapet in the direction of travel. For type “NY”, “W”, “M” or timber railings, name plate to be located on wing. For parapets, name plate to be located on inside face of parapet.

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.

The surface area for pile frictional computations is considered to be the projected “box area” of the H-pile, and not the actual steel surface area.

Clay is compressible to a far greater degree than sand or gravel. As the solid particles are pressed into closer contact with each other and water is squeezed out of the voids, only small frictional resistance to driving is generated because of the lubricating action of the free water. However, after driving is completed, the lateral pressure against the pile increases due to dissipation of the pore water pressures. This causes the fine clay particles to increase adherence to the comparatively rough surface of the pile. Load is transferred from the pile to the soil by the resulting strong adhesive bond. In many types of clay, this bond is stronger than the shearing resistance of the soil.

In hard, stiff clays containing a low percentage of voids and pore water, the compressibility is small. As a result, the amount of displacement and compression required to develop the pile’s full capacity are correspondingly small. As an H-pile is driven into stiff clay, the soil trapped between the flanges and web usually becomes very hard due to the compression and is carried down with it. This trapped soil acts as a plug and the pile can also act as a displacement pile.

In cases where loose soil is encountered, considerably longer point-bearing steel piles are required to carry the same load as relatively short displacement-type piles. This is because a displacement-type pile, with its larger cross section, produces more compaction as it is driven through materials such as soft clays or loose organic silts. H-piles are not typically used in exposed pile bents due to concerns with debris catchment.

11.3.1.12.3.2 Pipe Piles

Pipe piles consist of seamless, welded or spiral welded steel pipes in diameters ranging from 7-3/4 to 24 inches. Other sizes are available, but they are not commonly used. Typical wall thicknesses range from 0.375-inch to 0.75-inch, with wall thicknesses of up to 1.5 inches possible. Pipe piles should be specified by grade with reference to ASTM A 252.

Pipe piles may be driven either open or closed end. If the end bearing capacity from the full pile toe area is required, the pile toe should be closed with a flat plate or a conical tip.

11.3.1.12.3.3 Oil Field Piles

The oil industry uses a very high quality pipe in their drilling operations. Every piece is tested for conformance to their standards. Oil field pipe is accepted as a point-bearing alternative to HP piling, provided the material in the pipe meets the requirements of ASTM A 252, Grade 3, with a minimum tensile strength of 120 ksi or a Brinell Hardness Number (BHN) of 240, a minimum outside diameter of 7-3/4 inches and a minimum wall thickness of 0.375-inch. The weight and area of the pipe shall be approximately the same as the HP piling it replaces. Sufficient bending strength shall be provided if the oil field pipe is replacing HP piling in a pile bent. Oil field pipe is driven open-ended and not filled with concrete. The availability of this pile type varies and is subject to changes in the oil industry.

11.3.1.12.4 Pile Bents

See 13.1 for criteria to use pile bents at stream crossings. When pile bents fail to meet these criteria, pile-encased pier bents should be considered. To improve debris flow, round piles are generally selected for exposed bents. Round or H-piles can be used for encased bents.

11.3.1.13 Tolerable Movement of Substructures Founded on Driven Piles

WisDOT policy item:

For design of new bridge structures founded on driven piles, limit the horizontal movement at the top of the foundation unit to 0.5 inch or less at the service limit state.

11.3.1.14 Resistance Factors

The nominal (ultimate) geotechnical resistance capacity of the pile should be based on the type, depth and condition of subsurface material and ground water conditions reported in the Geotechnical Site Investigation Report, as well as the method of analysis used to determine pile resistance. Resistance factors to compute the factored geotechnical resistance are presented in **LRFD [Table 10.5.5.2.3-1]** and are selected based on the method used to determine the nominal (ultimate) axial compression resistance. The design intent is to adjust the resistance factor based on the reliability of the method used to determine the nominal pile resistance. When construction controls, are used to improve the reliability of capacity prediction (such as pile driving analyzer or static load tests), the resistance factors used during final design should be increased in accordance with **LRFD [Table 10.5.5.2.3-1]** to reflect planned construction monitoring.

WisDOT exception to AASHTO:

WisDOT requires at least four (4) piles per group to support each substructure unit, including each column for multi-column bents. WisDOT does not reduce geotechnical resistance factors to satisfy redundancy requirements to determine axial pile resistance. Hence, redundancy resistance factors in **LRFD [10.5.5.2.3]** are not applicable to WisDOT structures. This exception applies to typical CIP concrete pile and H-pile foundations. Non-typical foundations (such as drilled shafts) shall be investigated individually.

No guidance regarding the structural design of non-redundant driven pile groups is currently included in *AASHTO LRFD*. Since WisDOT requires a minimum of 4 piles per substructure unit, structural design should be based on a load modifier, η, of 1.0. Further description of load modifiers is presented in **LRFD [1.3.4]**.

The following geotechnical resistance factors apply to the majority of the Wisconsin bridges that are founded on driven pile. On the majority of WisDOT projects, wave equation analysis and dynamic monitoring are not used to set driving criteria. This equates to typical resistance factors of 0.35 to 0.45 for pile design. A summary of resistance factors is presented in Table 11.3-1, based on **LRFD [Table 11.5.5.2.3-1]**, which are generally used for geotechnical design on WisDOT projects.

<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 [5.5.4.2.1]**

- 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.

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

11.3.1.17.2 Lateral Squeeze

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

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

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

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

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

4. Use lightweight fill to reduce driving forces.

11.3.1.17.3 Uplift Resistance

Uplift forces may also be present, both permanently and intermittently, on a pile system. Such forces may occur from hydrostatic uplift or cofferdam seals, ice uplift resulting from ice grip on piles and rising water, wind uplift due to pressures against high structures or frost uplift. In the absence of pulling test data, the calculated factored shaft resistance should be used to determine static uplift capacity to demand ratio (CDR). A minimum CDR value of 1.0 is required. Generally, the type of pile with the largest perimeter is the most efficient in resisting uplift forces.

11.3.1.17.4 Pile Setup and Relaxation

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

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

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

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

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

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

11.3.1.17.5 Drivability Analysis

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

Drivability is treated as a strength limit state. The geotechnical engineer will perform the evaluation of this limit state during design based on a preliminary dynamic analysis using wave

						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 [5.7.4.4-3]**. $k_C = 0.85$ (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: $Pn = (0.66^{\lambda} * Fe * As)$ **LRFD [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. $Pr = \phi * Pn$

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

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

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

   - $Rn_{dyn} = Pr / \varphi_{dyn}$

     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.

11.3.1.18.1 Pile Hammers

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

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

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

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

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

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

- The required final resistance capacity of the pile.

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

11.3.1.18.2 Driving Formulas

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

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

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

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

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

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

The following modified Gates formula is used by WisDOT:

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

Where:

R_R	=	Factored pile resistance (tons)
φ_{dyn}	=	Resistance factor = 0.50, as specified in Table 11.3-1
R_{ndr}	=	Nominal pile resistance measured during pile driving (tons)

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

11.3.1.19 Construction Monitoring for Economic Evaluation of Deep Foundations

The goal of the foundation design is to provide the most efficient and economical design for the subsurface conditions. The design of pile-supported foundations is influenced by the resistance factor, which is generally a function of pile resistance determination during installation. The discussion in 11.3.1.14 presents the definition of resistance factors.

The typical method for a majority of the Department's deep foundation substructures is using the modified Gates to determine the RDR and to use a resistance factor of 0.50 based on department research and past experience. A comparison should be made between the use of the modified Gates and the use of the PDA with CAPWAP or the use of the Static Pile Load Test and the PDA with CAPWAP to determine which method is the most economical.

There are two possible methods available to economically use the PDA with CAPWAP to determine the required driving resistance, which allows the use of a resistance factor of 0.65.

Method 1: Reduce the number of piles in the substructure by driving the piles to the same RDR as using the modified Gates, but then increasing the FACR used in design. This is possible because the department has set a maximum value on the RDR, which when converted to the FACR is less than the structural capacity of the piles. This is true for all H-piles, and for some CIP piles when the FACR is controlled by the maximum allowable compression stress during driving based on 90 percent of the specified yield stress of steel.

Method 2: Drive each pile to a lower RDR, which should result in a shorter pile length. The number of piles per substructure would remain the same. The design estimated pile lengths are a function of the assumed soil conditions and the required driving resistance. The as-built pile lengths are a function of the actual soil conditions encountered and the contractor's hammer selection.

The department recommends Method 1 when evaluating the potential economic benefits of using the PDA with CAPWAP, because of the difficultly in accurately predicting pile lengths.

The method used to compare modified Gates to Static Pile Load Test(s) and the PDA with CAPWAP, which allows the use of a resistance factor of 0.80, would follow the procedures described in Method 1 used in the PDA with CAPWAP, reducing the number of piles per substructure. The number of static load test(s) will be a function of the size and number of substructures, the general spatial extent of the area in question and the variability of the subsurface conditions in the area of interest.

The costs to be included in the economic evaluation include the cost of the piling, the cost for the Department/Consultant to monitor the test piles, the cost for the Consultant CAPWAP evaluation (the Department does not currently have this capability), the unit costs for the contractor's time for driving and redriving the test piles, and the cost for the static pile load test(s).

Once the investigation of the subsurface conditions has been completed the geotechnical engineer and the structure engineer should discuss the potential for cost savings by increasing the resistance factor. The Bureau of Structures, Geotechnical Engineering Unit and the Region should be included in the discussion and should be part of the decision. Generally, the larger the project, the greater the potential for significant savings. The Department has two PDA's; therefore, the project team should contact the Geotechnical Engineering Unit (608-246-7940) to evaluate resources prior to incorporation of an increased resistance factor in the foundation design. PDA monitoring may be completed by Department or consultant personnel.

The following two examples use Method 1 to illustrate the potential cost savings/expenses for PDA with CAPWAP:

Pier
Pier Example: 12 x 53 H-piles to an estimated length of 100 feet at a unit cost of $40/foot. (**Note:** It is realized that for pier design the number of piles is not exclusively related to the vertical load, but this example is simplified for illustrative purposes).
Modified Gates: RDR = 220 tons, FACR = 110 tons, Total Load on Pier = 3,500 tons, Number of Piles = 3,500 tons / 110 tons = 32 piles Pile Cost = 32 piles x 100 feet x $40/ft = $128,000 Total Cost = $128,000
PDA/CAPWAP: RDR = 220 tons, FACR = 143 tons, Total Load on Pier = 3,500 tons, Number of Piles = 3,500 tons / 143 tons = 25 piles Pile Cost = 25 piles x 100 feet x $40/ft = $100,000 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 = $103,000
PDA/CAPWAP Savings = $25,000/pier

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 [5.7 and 5.8]**. This includes evaluation of axial resistance, combined axial and flexure, shear and buckling. It is noted that the critical load case for combined axial and flexure may be a load case that results in the minimum axial load or tension.

11.3.2.2 Resistance Factors

Resistance factors for drilled shafts are presented in Table 11.3-7 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.

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

Table 11.3-7

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 [5.5.4.2.1]**.

11.3.2.3 Bearing Resistance

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

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

Because these resistances mobilize at different rates, it may be more appropriate to add the ultimate end bearing to that portion of the side resistance remaining at the end of bearing failure. Or it may be more appropriate to add the ultimate side resistance to that portion of the end bearing mobilized at side resistance failure. Note that consideration of deflection, which is a service limit state, may control over the axial geotechnical resistance since displacements required to mobilize the ultimate end bearing can be excessive. 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]**

Table of Contents

12.1 General

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

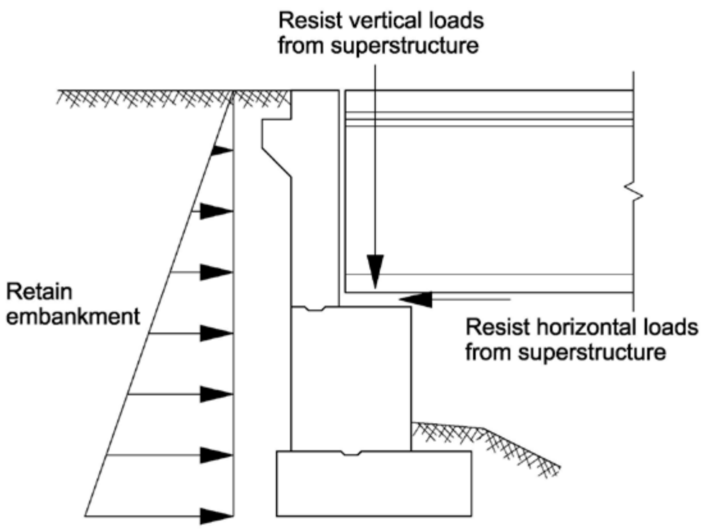


Figure 12.1-1
Primary Functions of an Abutment

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

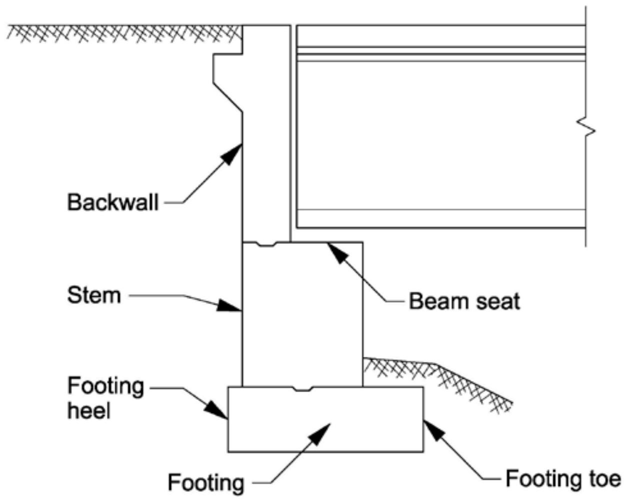


Figure 12.1-2
Components of an Abutment

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

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

12.2 Abutment Types

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

12.2.1 Full-Retaining

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

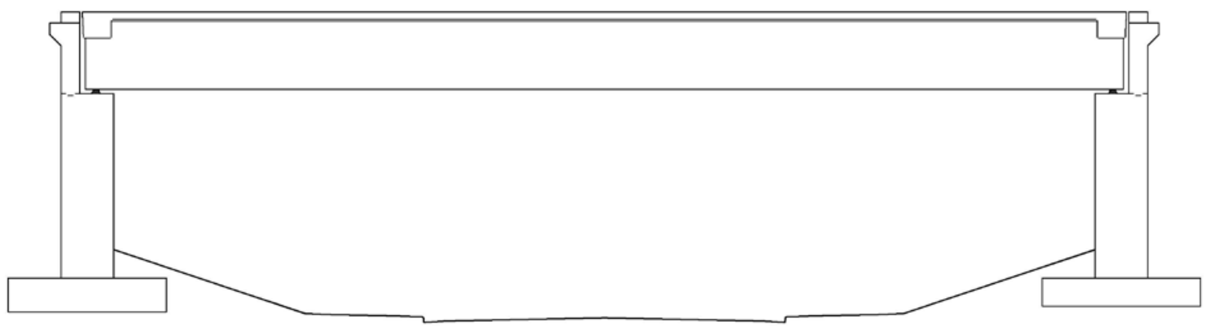


Figure 12.2-1
Full-Retaining Abutment

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

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

Other disadvantages of full-retaining abutments are:

- Minimum horizontal clearance

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

12.2.2 Semi-Retaining

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

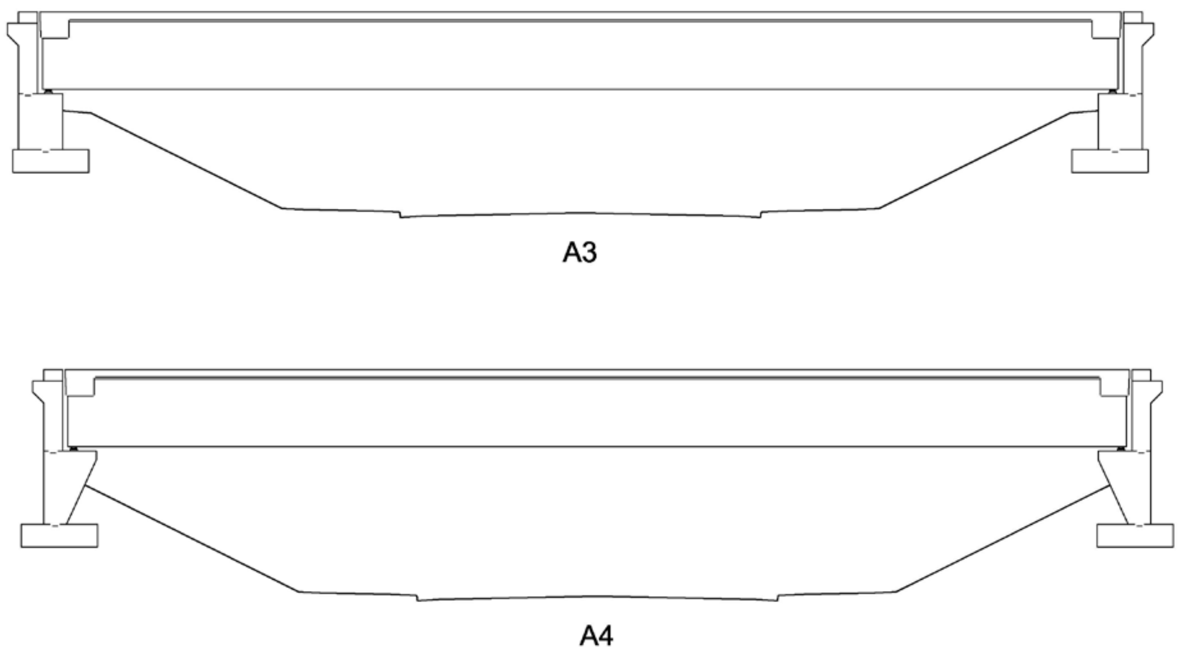


Figure 12.2-2
Semi-Retaining Abutment

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

Note: Type A4 abutments are currently under review and may be discontinued. Use of these abutments requires approval from the Bureau of Structures.

12.2.3 Sill

The sill abutment (Type A1) is constructed at the top of the slope after the roadway embankment is close to final grade, as shown in Figure 12.2-3. The sill abutment helps avoid many of the problems that cause rough approach pavements. It eliminates the difficulties of obtaining adequate compaction adjacent to the relatively high walls of closed abutments. Since the approach embankment may settle by forcing up or bulging up the slope in front of the abutment body, a berm is often constructed at the front of the body. The weight of the berm helps prevent such bulging.

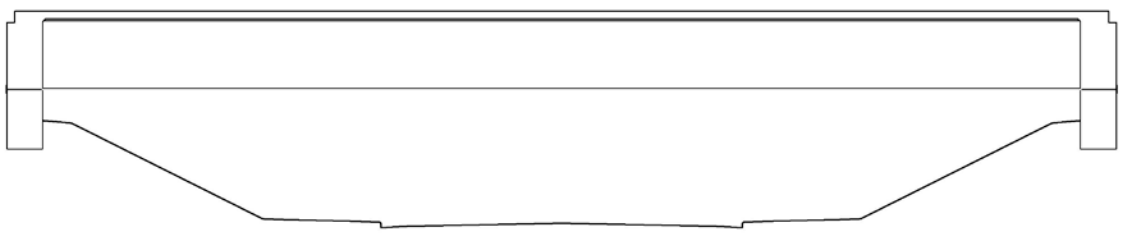


Figure 12.2-3
Sill Abutment

Sill abutments are the least expensive abutment type and are usually the easiest to construct. However, this abutment type results in a higher superstructure cost, so the overall cost of the structure should be evaluated with other alternatives.

For shallow superstructures where wing piles are not required, the Type A1 abutment is used with a fixed seat. This minimizes cracking between the body wall and wings. However, for shallow superstructures where wing piles are required, the Type A1 abutment is used with a semi-expansion seat. This allows superstructure movement, and it reduces potential cracking between the wings and body.

The parallel-to-abutment-centerline wings or elephant-ear wings, as shown on the Standard Details for Wings Parallel to A1 Abutment Centerline, should be used for grade separations when possible. This wing type is preferred because it increases flexibility in the abutment, it simplifies compaction of fill, and it improves stability. However, parallel-to-abutment-centerline wings should not be used for stream crossings when the high water elevation is above the bottom of the abutment. This wing configuration may not adequately protect bridge approaches and abutment backfill from the adjacent waterway.

12.2.4 Spill-Through or Open

A spill-through or open abutment is mostly used where an additional span may be added to the bridge in the future. It may also be used to satisfy unique construction problems. This abutment type is situated on columns or stems that extend upward from the natural ground. It is essentially a pier being used as an abutment.

It is very difficult to properly compact the embankment materials that must be placed around the columns and under the abutment cap. Early settlement and erosion are problems frequently encountered with spill-through or open abutments.

If the abutment is to be used as a future pier, it is important that the wings and backwall be designed and detailed for easy removal. Construction joints should be separated by felt or other acceptable material. Reinforcing steel should not extend through the joints. Bolts with threaded inserts should be used to carry tension stresses across joints.

12.2.5 Pile-Encased

Pile-encased abutments (Type A5) should only be used where documented cost data shows them to be more economical than sill abutments due to site conditions. For local roads right-of-way acquisition can be difficult, making the A5 a good option. Requiring crane access from only one side of a stream may be another reason to use a single span bridge with A5 abutments, as would savings in railing costs. Steeper topography may make A5 abutments a more reasonable choice than sill abutments. In general, however, using sill abutments with longer bridges under most conditions has cost advantages over using the Type A5 abutments. Type A5 abutments may require additional erosion control measures that increase construction cost.

The wall height of pile-encased abutments is limited to a maximum of 10 feet since increased wall height will increase soil pressure, resulting in uneconomical pile design due to size or spacing requirements. Reinforcement in the abutment body is designed based on live load surcharge and soil pressure on the back wall.

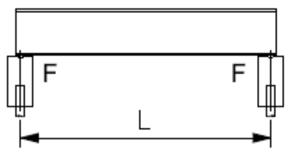
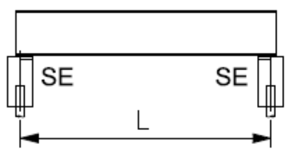
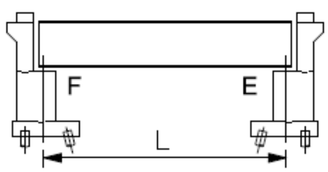
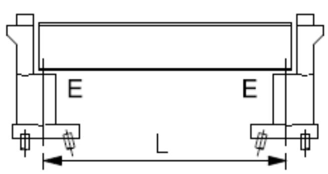
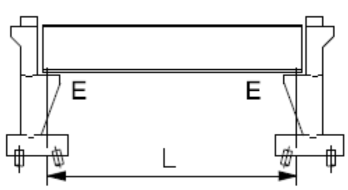
Pile-encased abutments with fixed seats are limited to a maximum skew of 15 degrees for girder structures and 30 degrees for slab structures in order to limit damage due to thermal expansion and contraction of the superstructure. Pile-encased abutments with a semi-expansion seat are limited to a maximum skew of 30 degrees. Wing skew angles are at 45 degrees relative to the body to prevent cracking between the abutment body and wings. These wings may be used for stream crossings when the high water elevation is above the bottom of the abutment. Parallel-to-roadway wings may be considered for extreme hydraulic conditions, however this will require a special design.

12.2.6 Special Designs

In addition to the standard abutment types described in the previous sections, many different styles and variations of those abutment types can also be designed. Such special abutment designs may be required due to special aesthetic requirements, unique soil conditions or unique structural reasons. Special designs of abutments require prior approval by the Bureau of Structures Development Chief.

12.7 Selection of Standard Abutment Types

From past experience and investigations, the abutment types presented in 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 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'	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
Type A4 (E-E) 	Not used	c. Based on geometry and economics Girder D < 60"	d. Based on geometry and economics Girder D < 60"
		REQUIRES APPROVAL BY THE BUREAU OF STRUCTURES	

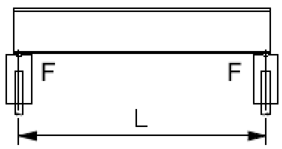
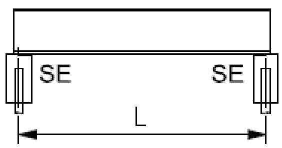
Abutment Arrangements	Superstructures		
	Concrete Slab Spans	Prestressed Girders	Steel Girders
Type A5 (F-F) 	L ≤ 150' S ≤ 30° AL ≤ 50'	L ≤ 150' S ≤ 15° AL ≤ 50'	L ≤ 150' S ≤ 15° AL ≤ 50'
Type A5 (SE-SE) 	L ≤ 200' S ≤ 30° AL > 50'	L ≤ 200' S ≤ 30°	L ≤ 150' S ≤ 30°
ABUTMENT TYPES			

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

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.

c.) For two-span prestressed girder bridges, the sill abutment is more economical than a semi-retaining abutment if the maximum girder length is not exceeded. It also is usually more economical if the next girder size is required.

d.) For two-span steel structures with long spans, the semi-retaining abutments may be more economical than sill abutments due to the shorter bridge lengths if a deeper girder is required.

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:

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

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

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

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

13.4.3 Vehicular Braking Force

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

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

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

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

13.4.4 Wind Loads

The design (3-second gust) wind speed (V) used in the determination of horizontal wind loads on superstructure and substructure units shall be taken from **LRFD [Table 3.8.1.1.2-1]**. The load combinations associated with the design of piers for wind load are Strength III, Strength V, and Service I. Their design wind speeds are:

- V =115 mph (Strength III)
- V = 80 mph (Strength V)
- V = 70 mph (Service I)

The wind pressure (P_Z) shall be determined as:

$P_Z = 2.56 \times 10^{-6} (V)^2 \cdot K_Z \cdot G \cdot C_D$ **LRFD [3.8.1.2.1]**

Where:

P_Z = design wind pressure (ksf)

V = design wind speed (mph) – (as stated above)

K_Z = pressure exposure and elevation coefficient

K_Z for Strength III is a function of ground surface roughness category as described in **LRFD [3.8.1.1.4]** and wind exposure category as described in **LRFD [3.8.1.1.5, 3.8.1.1.3]**, and is determined using **LRFD [Eq'ns 3.8.1.2.1-2**, **3.8.1.2.1-3**, **or 3.8.1.2.1-4]**.

- K_Z (Strength III) = see **LRFD [Table C3.8.1.2.1-1]**

K_Z for Strength V and Service I is not a function of bridge height, type, and wind exposure category **LRFD [3.8.1.2]**, and their values are:

- K_Z (Strength V) = 1.0
- K_Z (Service I) = 1.0

G = gust effect factor

- G (Strength III) = 1.0 **LRFD [Table 3.8.1.2.1-1]**
- G (Strength V) = 1.0 **LRFD [3.8.1.2.1]**
- G (Service I) = 1.0 **LRFD [3.8.1.2.1]**

C_D = drag coefficient for Strength III, Strength V, Service I **LRFD [Table 3.8.1.2.1-2]**

- C_D (girder/slab -superstructure) = 1.3
- C_D (substructure) = 1.6

Substituting these values into the equation for wind pressure (P_Z) gives:

- Strength III – P_Z (girder/slab -superstructure) = 0.044 · (K_Z) ksf
  P_Z (substructure) = 0.054 · (K_Z) ksf
- Strength V – P_Z (girder/slab -superstructure) = 0.021 ksf
  P_Z (substructure) = 0.026 ksf
- Service I – P_Z (girder/slab -superstructure) = 0.016 ksf
  P_Z (substructure) = 0.020 ksf

Wind pressure shall be assumed to be uniformly distributed on the area exposed to the wind. The exposed area shall be the sum of the area of all components as seen in elevation taken perpendicular to the wind direction. See 13.4.4.1 and 13.4.4.2 for additional information regarding application of these wind pressures.

Wind loads are divided into the following four types.

13.4.4.1 Wind Load from the Superstructure

The transverse and longitudinal wind load (WS_{SUPER}) components transmitted by the superstructure to the substructure for various angles of wind direction may be taken as the product of the skew coefficients specified in **LRFD [Table 3.8.1.2.3a-1]**, the wind pressure (P_Z) calculated as shown in 13.4.4, and the depth of the superstructure, as specified in **LRFD [3.8.1.2.3a]**. The depth shall be as seen in elevation perpendicular to the longitudinal axis of the bridge.

Both components of the wind loads shall be applied as line loads. The longitudinal component shall be applied at the bearing elevation, and the transverse component shall be applied at the mid-depth of the superstructure. In plan, the longitudinal components of wind loads shall be applied as line loads along the longitudinal axis of the superstructure. The purpose of applying the line load along the longitudinal axis of the bridge in plan is to avoid introducing a moment in the horizontal plane of the superstructure. The skew angle shall be taken as measured from the perpendicular to the longitudinal axis of the bridge in plan. Wind direction for design shall be that which produces the maximum force effect in the substructure. The transverse and longitudinal wind load components on the superstructure shall be applied simultaneously.

For girder bridges, the wind loads may be taken as the product of the wind pressure, skew coefficients, and the depth of the superstructure including the depth of the girder, deck, floor system, parapet, and sound barrier. Do not apply wind pressure to open rails or fences. Do apply wind pressure to all parapets, including parapets located between the roadway and the sidewalk if there is an open rail or fence on the edge of the sidewalk.

For usual girder and slab bridges having an individual span length of not more than 150 ft. (160 ft. for prestressed girders) and a maximum height of 33 ft. above low ground or water level, the following simplified method for wind load components (WS_{SUPER}) may be used:

- Transverse: 100% of the wind load calculated based on wind direction perpendicular to the longitudinal axis of the bridge.
- Longitudinal: 25% of the transverse load.

The wind load components are to be multiplied by the tributary length that the wind load is applied to, as described in 13.5, to produce the wind forces in the transverse and longitudinal directions.

Both forces shall be applied simultaneously.

WisDOT policy item:

For usual girder and slab bridges having an individual span length of not more than 150 ft. (160 ft. for prestressed girders) and a maximum height of 33 ft. above low ground or water level, the following simplified method for wind load components (WS_{SUPER}) may be used:

Strength III:

- 0.044 ksf, transverse
- 0.011 ksf, longitudinal

Strength V:

- 0.021 ksf, transverse
- 0.006 ksf, longitudinal

Service I:

- 0.016 ksf, transverse
- 0.004 ksf, longitudinal

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

13.4.4.2 Wind Load Applied Directly to Substructure

The transverse and longitudinal wind loads (WS_{SUB}) to be applied directly to the substructure shall be calculated using the wind pressure (P_Z) determined as shown in 13.4.4, and as specified in **LRFD [3.8.1.2.3b]**. For wind directions taken skewed to the substructure, the wind pressure shall be resolved into components perpendicular to the end and front elevations of the substructure. The component perpendicular to the end elevation shall act on the exposed substructure area as seen in end elevation, and the component perpendicular to the front elevation shall act on the exposed substructure area as seen in the front elevation. The resulting wind forces shall be taken as the product of the value of resolved (P_Z) components acting on the end and front elevations, times its corresponding exposed area. These forces are applied at the centroid of the exposed area. The two substructure wind force components shall be applied simultaneously with the wind forces from the superstructure.

When combining the wind forces applied directly to the substructure with the wind forces transmitted to the substructure from the superstructure, all wind forces should correspond to wind blowing from the same direction.

WisDOT policy item:

The following conservative values for wind applied directly to the substructure, (WS_{SUB}), may be used for all bridges:

Strength III:

- 0.054 ksf, transverse
- 0.054 ksf, longitudinal

Strength V:

- 0.026 ksf, transverse
- 0.026 ksf, longitudinal

Service I:

- 0.020 ksf, transverse
- 0.020 ksf, longitudinal

Both forces shall be applied simultaneously.

13.4.4.3 Wind Load on Vehicles

Wind load on live load (WL) shall be represented by an interruptible, moving force of 0.10 klf acting transverse to, and 6.0 ft. above, the roadway and shall be transmitted to the structure as specified in **LRFD [3.8.1.3]**. The load combinations that are associated with this load are Strength V and Service I.

For various angles of wind direction, the transverse and longitudinal components of the wind load on live load may be taken as specified in **LRFD [Table 3.8.1.3-1]** with the skew angle measured from the perpendicular to the longitudinal axis of the bridge in plan.

The wind direction for design shall be that which produces the extreme force effect on the component under investigation. The transverse and longitudinal wind load components on the live load shall be applied simultaneously. The longitudinal component shall be applied at the bearing elevation, and the transverse component shall be applied at 6.0 ft. above the roadway surface.

For usual girder and slab bridges having an individual span length of not more than 150 ft. (160 ft. for prestressed girders) and a maximum height of 33 ft. above low ground or water level, the following simplified method for wind load components on live load (WL) may be used:

- 0.10 klf , transverse (Strength V, Service I)
- 0.04 klf , longitudinal (Strength V, Service I)

The wind load components are to be multiplied by the tributary length that the wind load is applied to, as described in 13.5, to produce the wind forces in the transverse and longitudinal directions.

Both forces shall be applied simultaneously.

This horizontal wind load (WL) should be applied only to the tributary lengths producing a force effect of the same kind, similar to the design lane load. These loads are applied in conjunction with the horizontal wind loads described in 13.4.4.1 and 13.4.4.2.

13.4.4.4 Vertical Wind Load

The effect of wind forces tending to overturn structures, unless otherwise determined according to **LRFD [3.8.3]**, shall be calculated as a vertical upward wind load (WS_{VERT}) as specified in **LRFD [3.8.2]**, and shall be equal to:

- 0.020 ksf (Strength III)

times the width of the deck, including parapets and sidewalks, and shall be applied as a longitudinal line load. This load shall be applied only when the direction of horizontal wind is taken to be perpendicular to the longitudinal axis of the bridge. This line load shall be applied at the windward ¼ point of the deck width, which causes the largest upward force at the windward fascia girder. This load is applied in conjunction with the horizontal wind loads described in 13.4.4.1 and 13.4.4.2.

WisDOT policy item:

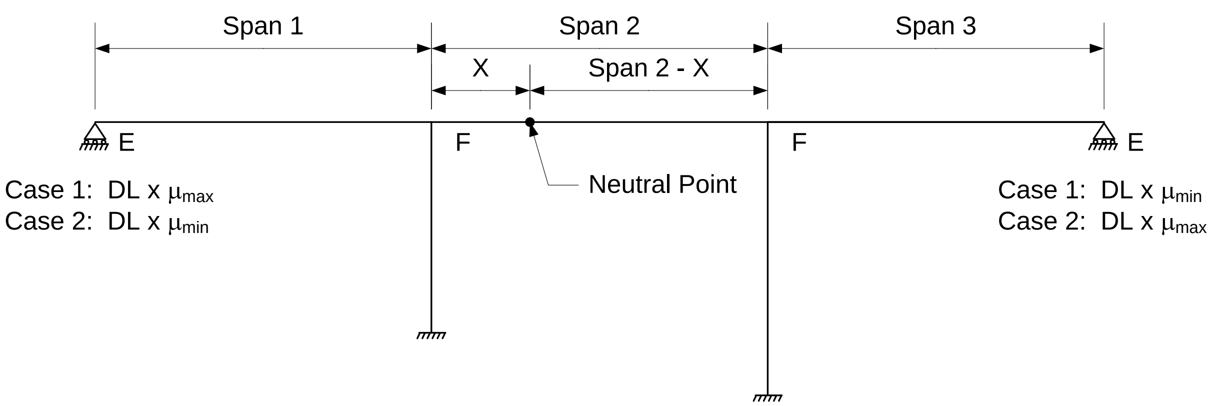
If WisDOT policy items are being applied in 13.4.4.1 and 13.4.4.2, assume the wind direction is perpendicular to the longitudinal axis of the bridge and apply the vertical wind load as described above.

The vertical wind load (WS_{VERT}) is applied with load combinations that do not involve wind on live load, because the high wind velocity associated with this load would limit vehicles on the bridge, such as for load combination Strength III. The wind load shall be multiplied by the tributary length that the wind load is applied to, as described in 13.5, to produce the vertical wind force.

13.4.5 Uniform Temperature Forces

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

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



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

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

Figure 13.4-1
Neutral Point Location with Multiple Fixed Piers

As used in Figure 13.4-1:

E = Column or shaft modulus of elasticity (ksi)

I = Column or shaft gross moment of inertia about longitudinal axis of the pier (in^4)

α = Superstructure coefficient of thermal expansion (ft/ft/°F)

T = Temperature change of superstructure (°F)

μ = Coefficient of friction of the expansion bearing (dimensionless)

h = Column height (ft)

DL = Total girder dead load reaction at the bearing (kips)

X = Distance between the fixed pier and the neutral point (ft)

The temperature force on a single fixed pier in a bridge is the resultant of the unbalanced forces acting on the substructure units. Maximum friction coefficients are assumed for expansion bearings on one side of the pier and minimum coefficients are assumed on the other side to produce the greatest unbalanced force on the fixed pier. For bridges with only one pier (fixed), do not include temperature force, TU, in the design of the pier when the abutments are either fixed or semi-expansion.

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

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

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

Where:

L = Superstructure expansion length between neutral point and location being considered (ft)

F = Force per column applied at the bearing elevation (kips)

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

The values for computing temperature forces in Table 13.4-1 shall be used on Wisconsin bridges. Do not confuse this temperature change with the temperature range used for expansion joint design.

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

Table 13.4-1
Temperature Expansion Values

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

13.4.6 Force of Stream Current

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

13.4.6.1 Longitudinal Force

The longitudinal force is computed as follows:

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

Where:

p = Pressure of flowing water (ksf)

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

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

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

13.4.6.2 Lateral Force

The lateral force is computed as follows:

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

Where:

p = Lateral pressure of flowing water (ksf)

C_D = Lateral drag coefficient (dimensionless), as presented in Table 13.4-2

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

Table 13.4-2
Lateral Drag Coefficient Values

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

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

13.4.7 Buoyancy

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

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

The submerged weight of the soil above the footing is used for calculating the vertical load on the footing. Typical values are presented in Table 13.4-3.

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

Table 13.4-3
Submerged Unit Weights of Various Soils

13.4.8 Ice

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

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

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

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

13.4.8.1 Force of Floating Ice and Drift

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

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

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

$$\mathrm{F} = \mathrm{F_c} = \mathrm{C_a} \cdot \mathrm{p} \cdot \mathrm{t} \cdot \mathrm{w}$$

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

Where:

p = Effective ice crushing strength (ksf)

t = Ice thickness (ft)

w = Pier width at level of ice action (ft)

WisDOT policy item:

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

WisDOT policy item:

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

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

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

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

WisDOT exception to AASHTO:

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

13.4.8.2 Force Exerted by Expanding Ice Sheet

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

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

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

13.4.9 Centrifugal Force

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

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

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

Where:

V	=	Highway design speed (ft/sec)
g	=	Gravitational acceleration = 32.2 (ft/sec^2)
R	=	Radius of curvature of travel lane (ft)

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

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

13.4.10 Extreme Event Collision Loads

WisDOT policy item:

With regards to **LRFD [3.6.5]** and vehicular collision force, CT, protecting the pier and designing the pier for the 600 kip static force are each equally acceptable. The bridge design engineer should work with the roadway engineer to determine which alternative is preferred.

WisDOT policy item:

Designs for bridge piers adjacent to roadways with a design speed ≤ 40 mph need not consider the provisions of **LRFD [3.6.5]**.

If the design speed of a roadway adjacent to a pier is > 40 mph and the pier is not protected by a TL-5 barrier, embankment or adequate offset, the pier columns/shaft, *only*, shall be strengthened to comply with **LRFD [3.6.5]**. For a multi-column pier the minimum size column shall be 3x4 ft rectangular or 4 ft diameter (consider clearance issues and/or the wide cap required when using 4 ft diameter columns). Solid shaft and hammerhead pier shafts are considered adequately sized.

All multi-columned piers require a minimum of three columns. If a pier cap consists of two or more segments each segment may be supported by two columns. If a pier is constructed in stages, two columns may be used for the temporary condition.

The vertical reinforcement for the columns/shaft shall be the greater of what is required by design (not including the Extreme Event II loading) or a minimum of 1.0% of the gross concrete section (total cross section without deduction for rustications less than or equal to 1-1/2" deep) to address the collision force for the 3x4 ft rectangular and 4 ft diameter columns.

For the 3x4 ft rectangular columns, use double #5 stirrups spaced at 6" vertically as a minimum. For the 4 ft diameter columns, use #4 spiral reinforcement (smooth bars) spaced vertically at 6" as a minimum. Hammerhead pier shafts shall have, as a minimum, the horizontal reinforcement as shown on the Standards.

See Standard for Multi-Columned Pier with Rectangular Columns for an acceptable design to meet **LRFD [3.6.5]**.

WisDOT exception to AASHTO:

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

13.5 Load Application

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

13.5.1 Loading Combinations

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

Load Combination Limit State	Load Factor										
	DC		DW		LL+IM BR CE	WA	WS	WL	FR	TU CR SH	IC CT CV
	Max.	Min.	Max.	Min.							
Strength I	1.25	0.90	1.50	0.65	1.75	1.00	0.00	0.00	1.00	0.5*	0.00
Strength III	1.25	0.90	1.50	0.65	0.00	1.00	1.00	0.00	1.00	0.5*	0.00
Strength V	1.25	0.90	1.50	0.65	1.35	1.00	1.00	1.00	1.00	0.5*	0.00
Service I	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
Extreme Event II	1.25	0.90	1.50	0.65	0.50	1.00	0.00	0.00	1.00	0.00	1.00

Table 13.5-1
Load Factors

* Values based on using gross moment of inertia for analysis **LRFD [3.4.1]**

13.5.2 Expansion Piers

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

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

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

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

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

Where:

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

Example E27-1.8 illustrates the calculation of this force.

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

13.5.3 Fixed Piers

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

For fixed bearings, longitudinal forces, other than temperature, are based on loading one-half of the adjacent span lengths. If longitudinal forces, other than temperature, at expansion substructure units exceed the maximum friction value of the bearings, the fixed piers need to assume the additional force beyond the maximum friction. The longitudinal forces are applied at the bearing elevation.

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

13.6 Multi-Column Pier and Cap Design

WisDOT policy item:

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

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

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

WisDOT policy item:

To reduce excessive thermal and/or shrinkage forces, pier caps greater than 65' long may be made non-continuous. Each segment may utilize as few as two columns. Spacing between ends of adjacent cap segments shall be 1'-0" minimum.

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

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

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

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

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

13.7 Hammerhead Pier Cap Design

WisDOT policy item:

Hammerhead pier caps shall be designed using the strut-and-tie method **LRFD [5.6.3]**.

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 [5.6.3.1]**. 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.

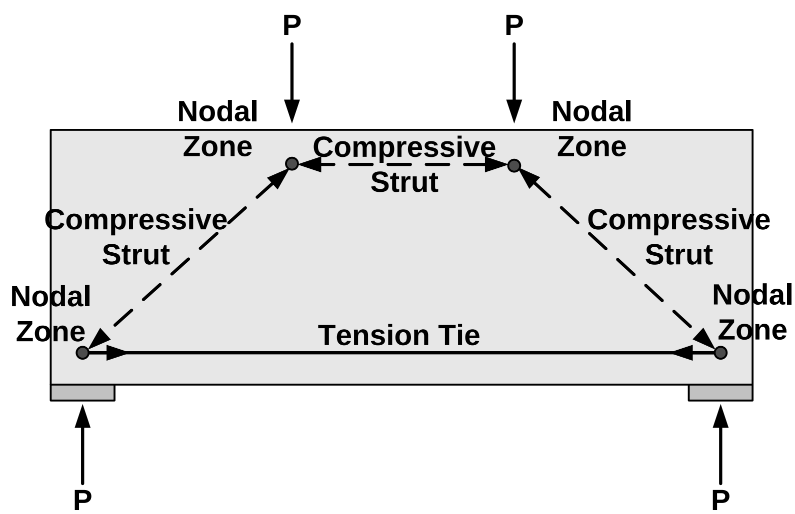


Figure 13.7-1
Basic Strut-and-Tie Elements

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 [5.6.3.2]**. 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.

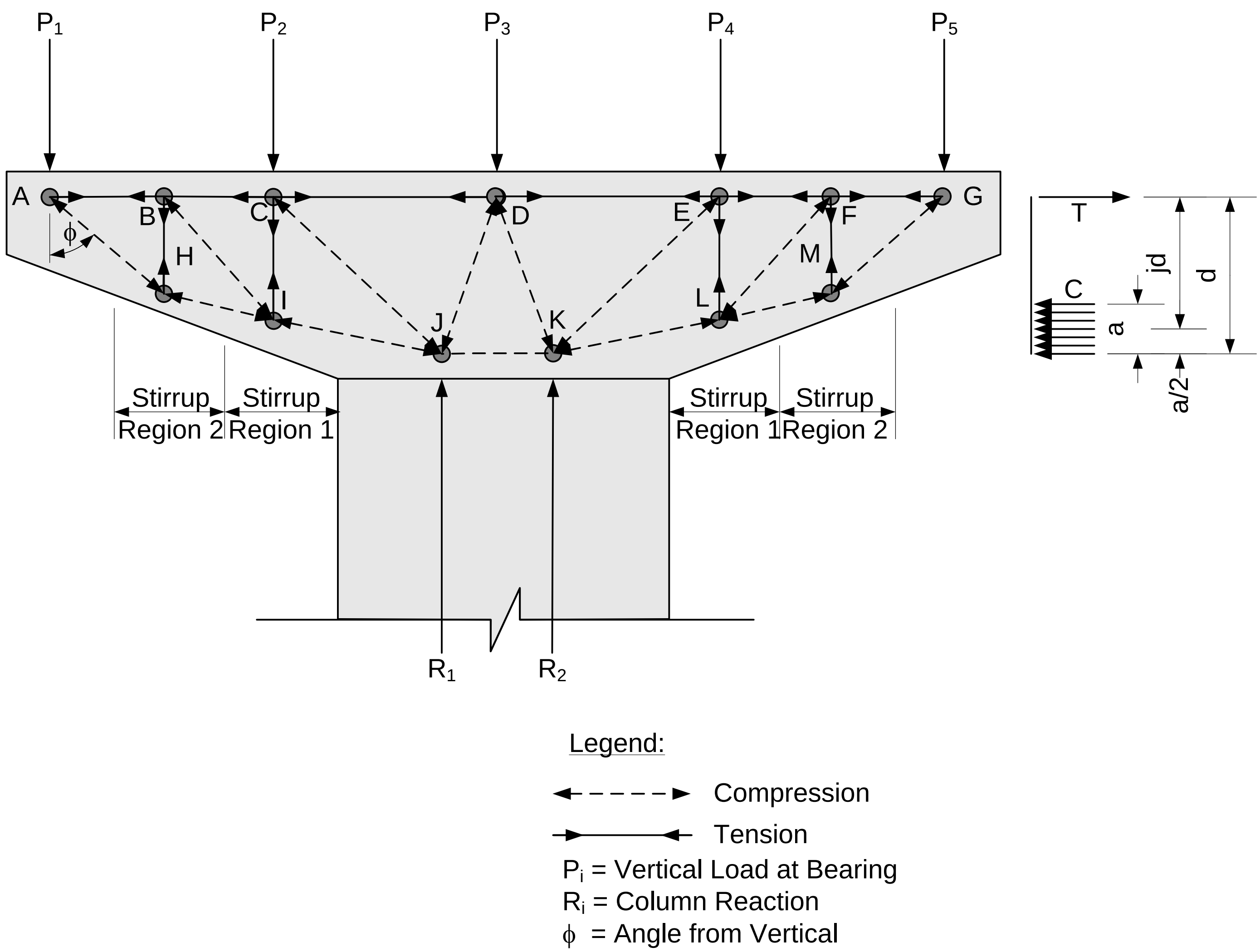


Figure 13.7-2
Example Hammerhead Pier Cap Strut-and-Tie Model

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:

- <u>CCC</u>: Nodes where only struts intersect

- <u>CCT</u>: 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 [5.6.3.5]**, the concrete area supporting the bearing devices shall satisfy the following:

$P_u \leq \phi \cdot P_n$ **LRFD [5.6.3.3]**

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 [5.6.3.5.1]**

Where:

f_{cu} = Limiting compressive stress at the face of a node **LRFD [5.6.3.5.3]** (ksi)

A_{cn} = Effective cross-sectional area of the node face **LRFD [5.6.3.5.2]** (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 [5.7.5]**

ν = Concrete efficiency factor (0.45, when no crack control reinforcement is present ; see **LRFD [Table 5.6.3.5.3a-1]** for values when crack control reinforcement is present per **LRFD [5.6.3.6]**)

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 [5.6.3.6]**:

$A_{brg} \geq P_u / \phi \cdot (m \cdot 0.85 \cdot f'_c)$ --- (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 [5.6.3.4.1]**. For non-prestressed caps, the tension tie steel shall satisfy:

$P_u \leq \phi \cdot P_n$ **LRFD [5.6.3.3]**

$P_n = f_y \cdot A_{st}$; 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 [5.6.3.4.1]**. 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 [5.7.3.4]**. 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 [5.6.3.5]**.

$P_u \leq \phi \cdot P_n$ **LRFD [5.6.3.3]**

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

$P_n = f_{cu} \cdot A_{cn}$ **LRFD [5.6.3.5.1]** --- (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 [5.6.3.5.3]** (ksi)
A_{cn}	=	Effective cross-sectional area of the node face at the strut **LRFD [5.6.3.5.2]** (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 5.6.3.5.3a-1]** for values when crack control reinforcement is present per **LRFD [5.6.3.6]**)

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 [5.6.3.6]**:

$P_u \le \phi \cdot (0.65 \cdot f'_c) \cdot A_{cn}$ --- (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}.

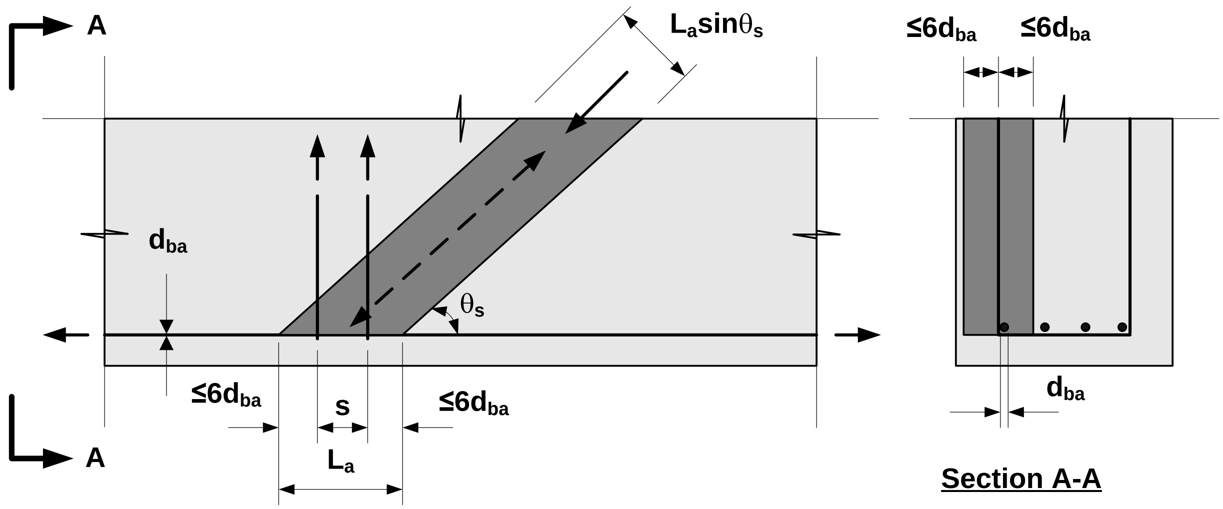


Figure 13.7-3
Strut Anchored by Tension Reinforcement Only (CTT)

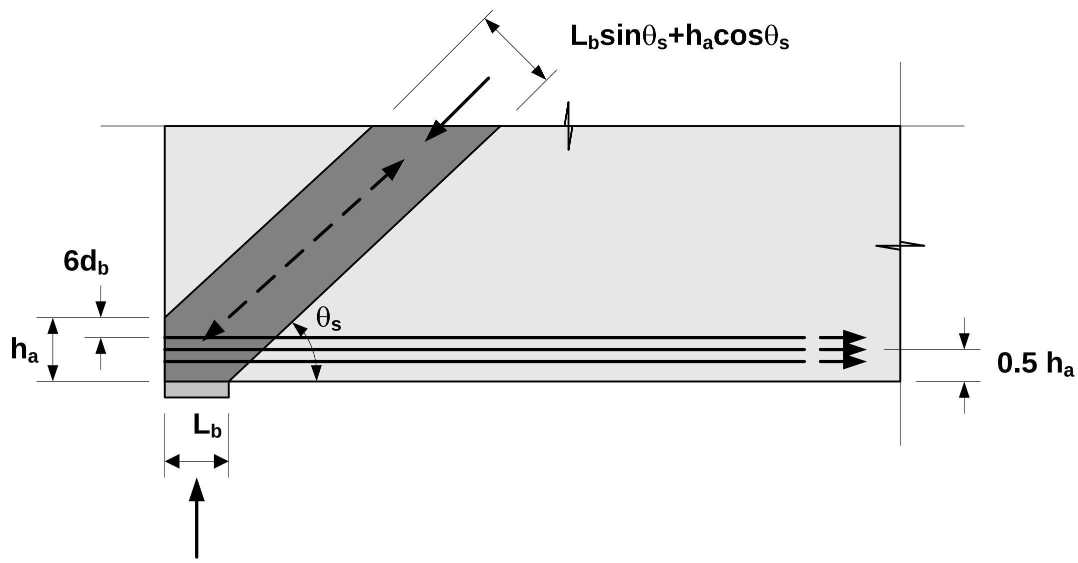


Figure 13.7-4
Strut Anchored by Bearing and Tension Reinforcement (CCT)

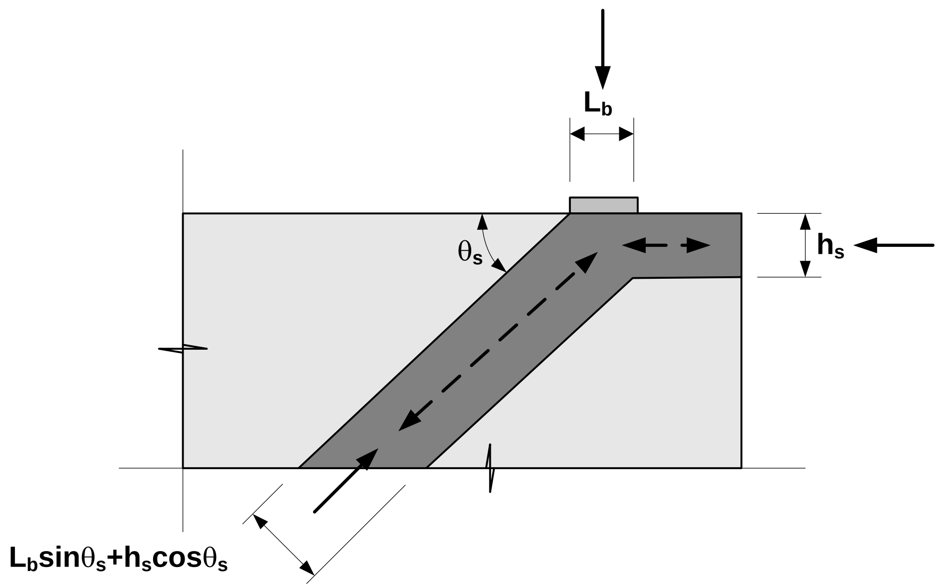


Figure 13.7-5
Strut Anchored by Bearing and Strut (CCC)

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 [5.6.3.4.2]**, 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 [5.11]**.

13.7.7 Provide Crack Control Reinforcement

Pier caps designed using the strut-and-tie method and the efficiency factors of **LRFD [Table 5.6.3.5.3a-1]**, shall contain an orthogonal grid of reinforcing bars near each face in accordance with **LRFD [5.6.3.6]**. 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 [5.7.3.4]** 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 [5.7.4]**. The minimum allowable column size is 2'-6" in diameter. The minimum steel bar area is as specified in **LRFD [5.7.4.2]**. 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 [5.7.4.3]**. 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 [5.7.4.1]** 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 [5.7.3.4]** shall be considered for pier columns. All pier columns shall be designed using a Class 2 exposure condition exposure factor.

13.10 Pile Bent and Pile Encased Pier Analysis

WisDOT policy item:

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

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

WisDOT policy item:

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

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

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

Where:

F = Total factored vertical load (kips)

n = Number of piles

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

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

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

In which:

d = Distance of pile from pile group centroid

c = Distance from outermost pile to pile group centroid

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

13.11 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 [5.7.3.4]** shall not be considered for pier isolated spread footings, isolated pile footings and continuous footings.

Shrinkage and temperature reinforcement, as defined in **LRFD [5.10.8]** 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 [5.13.3]**. 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 [5.7.3]**. 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.

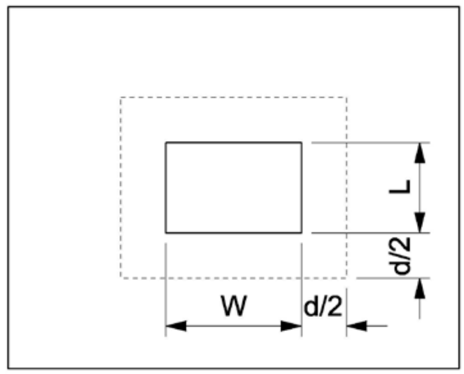


Figure 13.11-1
Critical Perimeter Location for Spread Footings

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.

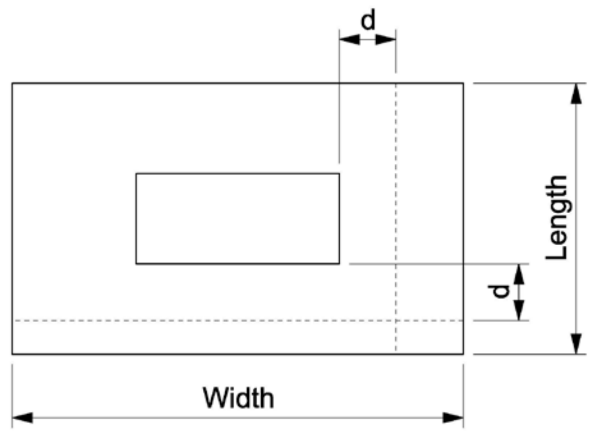


Figure 13.11-2
Shear Location for One-Way Action

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 [5.13.3]**. 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 [5.7.3]**. 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 π(2R + d) for round columns. The critical perimeter location for pile footings with rectangular columns is illustrated in Figure 13.11-3.

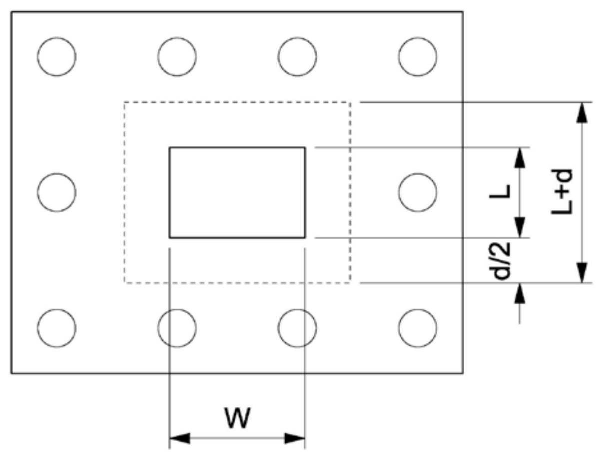


Figure 13.11-3
Critical Perimeter Location for Pile Footings

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.

Application	Value of Bond
Bond on Piles	10 psi
Bond on Sheet Piling	2 psi applied to [(Seal Depth - 2’) x Seal Perimeter]

Table 13.11-1
Bond on Piles and Sheet Piling

Lateral forces from stream flow pressure are resisted by the penetration of the sheet piling below the streambed elevation and by the bracing inside the cofferdam. When seals for spread footings are founded on rock, the weight of the seal is used to counterbalance the lateral stream flow pressure.

The downstream side of the cofferdam should be keyed into rock deep enough or other measures should be used to resist the lateral stream flow pressure. To provide a factor of safety, the cofferdam weight (sheet piling and bracing) is ignored in the analysis. The design stream flow velocity is based on the flow at the site at the time of construction but need not exceed 75% of the 100-year velocity. The force is calculated as per 13.4.6.

A rule of thumb for seal thickness is 0.40H for spread footings and 0.25H for pile footings, where H is the water depth from bottom of seal to top of water. The 2-year high water elevation, if available, should be used as the estimated water elevation during construction. The assumed water elevation used to determine the seal thickness should be noted on the plans. The minimum seal size is 3’-0” larger than the footing size on all sides. See Standard for Hammerhead Pier for additional guidance regarding the sizing of the seal.

Example: Determine the seal thickness for a 9' x 12' footing with 12-12" diameter piles. Uplift capacity of one pile equals 15 kips (per the Geotechnical Engineer). The water depth to the top of seal is 16'.

Assume 15' x 18' x 3.25' seal.

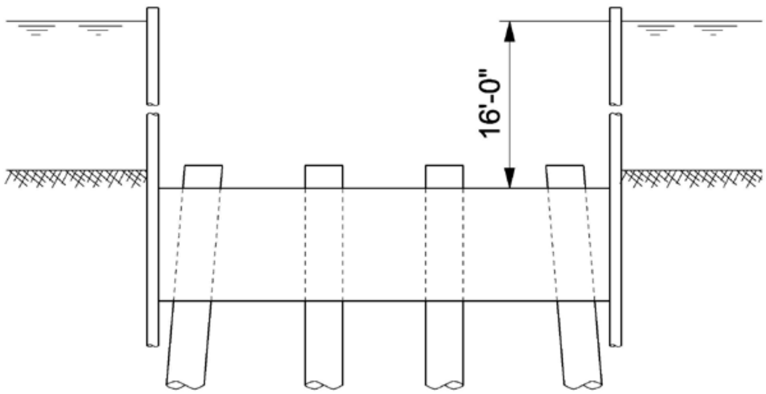


Figure 13.11-4
Seal Inside a Cofferdam

Uplift force of water	15 x 18 x 19.25 x 0.0624	=	324.3 kips (up)
Weight of seal course	15 x 18 x 3.25 x 0.15	=	131.6 kips (down)
Friction of sheet piling	2 x (15+18) x (3.25 - 2.0) x 144 x 0.002	=	23.8 kips (down)
Pile frictional resistance	π x 12 x (3.25 x 12) x 0.010	=	14.7 kips
Pile uplift resistance	(Per Geotechnical Engineer)	=	15.0 kips
Total pile resistance	12 piles x min(14.7,15.0)	=	176.4 kips (down)
Sum of downward forces	131.6+23.8+176.4	=	332 kips
Sum of upward forces	324.3	=	324 kips

332 > 324 OK

USE 3'- 3" THICK SEAL

Note: Pile uplift resistance shall be determine by the Geotechnical Engineer. For this example, when the pile uplift resistance equals 10 kips a 4'-6" thick seal is required.

13.12 Quantities

Consider the "Upper Limits for Excavation" for piers at such a time when the quantity is a minimum. This is either at the existing ground line or the finished graded section. Indicate in the general notes which value is used.

Structure backfill is not used at piers except under special conditions.

Compute the concrete quantities for the footings, columns and cap, and show values for each of them on the final plans.

13.13 Design Examples

E13-1 Hammerhead Pier Design Example

E13-2 Multi-Column Pier Design Example

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E13-1 Hammerhead Pier Design Example

This example shows design calculations conforming to the ***AASHTO LRFD Bridge Design Specifications (Seventh Edition - 2016 Interim)*** as supplemented by the *WisDOT Bridge Manual* The design methods presented throughout the example are meant to be the most widely used in general bridge engineering practice.

The first design step is to identify the appropriate design criteria. This includes, but is not limited to, defining material properties, identifying relevant superstructure information, and determining the required pier geometry.

E13-1.1 Obtain Design Criteria

This pier is designed for the superstructure as detailed in example **E24-1**. This is a two-span steel girder stream crossing structure. Expansion bearings are located at the abutments, and fixed bearings are used at the pier.

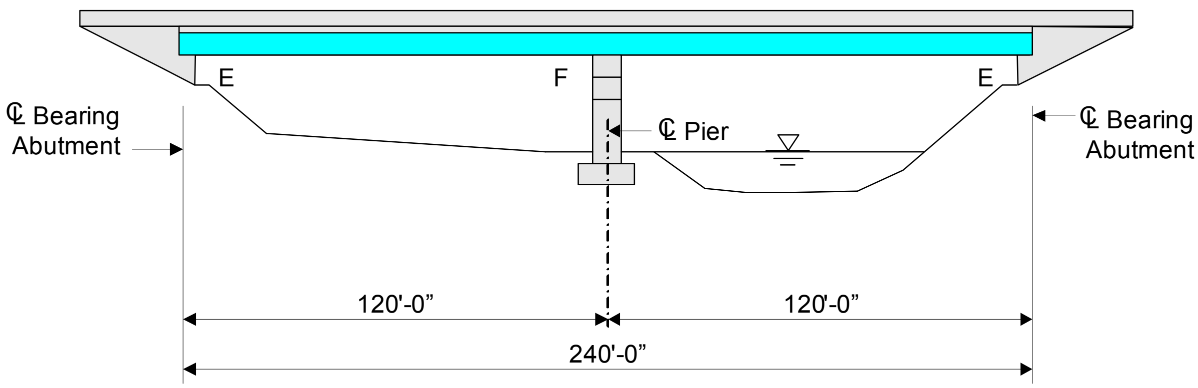


Figure E13-1.1-1
Bridge Elevation

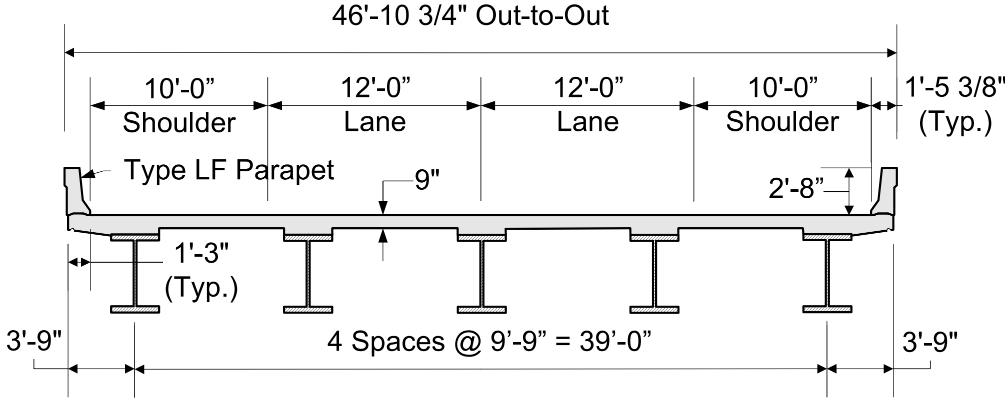


Figure E13-1.1-2
Bridge Cross Section

E13-1.1.1 Material Properties:

$w_c := 0.150$	unit weight of concrete, kcf
$f'_c := 3.5$	concrete 28-day compressive strength, ksi
$f_y := 60$	reinforcement yield strength, ksi

E13-1.1.2 Reinforcing Steel Cover Requirements:

All cover dimensions listed below are in accordance with **LRFD [Table 5.12.3-1]** and are shown in inches.

$Cover_{cp} := 2.5$	Pier cap
$Cover_{co} := 2.5$	Pier column
$Cover_{ft} := 2.0$	Footing top cover
$Cover_{fb} := 6.0$	Footing bottom cover, based on standard pile projection

E13-1.2 Relevant Superstructure Data

$w_{deck} := 46.50$	Deck Width, ft
$w_{roadway} := 44.0$	Roadway Width, ft
$ng := 5$	Number of Girders
$S := 9.75$	Girder Spacing, ft
$DOH := 3.75$	Deck Overhang, ft (Note that this overhang exceeds the limits stated in Chapter 17.6.2. WisDOT practice is to limit the overhang to 3'-7".)
$N_{spans} := 2$	Number of Spans
$L := 120.0$	Span Length, ft
$skew := 0$	Skew Angle, degrees
$H_{super} = 8.46$	Superstructure Depth, ft
$H_{brng} := 6.375$	Bearing Height, in (Fixed, Type A)
$W_{brng} := 18$	Bearing Width, in
$L_{brng} := 26$	Bearing Length, in
$\mu_{max} := 0.10$	Max. Coefficient of Friction of Abutment Expansion Bearings
$\mu_{min} := 0.06$	Min. Coefficient of Friction of Abutment Expansion Bearings

E13-1.2.1 Girder Dead Load Reactions

Unfactored Dead Load Reactions, kips

$$DLR_{int} := \begin{pmatrix} \text{"LoadType"} & \text{"Abut"} & \text{"Pier"} \\ \text{"Beam"} & 7.00 & 34.02 \\ \text{"Misc"} & 1.23 & 4.73 \\ \text{"Deck"} & 46.89 & 178.91 \\ \text{"Parapet"} & 6.57 & 24.06 \\ \text{"FWS"} & 7.46 & 27.32 \end{pmatrix} \qquad DLR_{ext} := \begin{pmatrix} \text{"LoadType"} & \text{"Abut"} & \text{"Pier"} \\ \text{"Beam"} & 7.00 & 34.02 \\ \text{"Misc"} & 0.83 & 3.15 \\ \text{"Deck"} & 48.57 & 185.42 \\ \text{"Parapet"} & 6.57 & 24.06 \\ \text{"FWS"} & 7.46 & 27.32 \end{pmatrix}$$

Abutment Reactions:

$\boxed{AbutRint_{DC} = 61.69}$ kips $\boxed{AbutRext_{DC} = 62.97}$ kips

$\boxed{AbutRint_{DW} = 7.46}$ kips $\boxed{AbutRext_{DW} = 7.46}$ kips

Pier Reactions:

$\boxed{Rint_{DC} = 241.72}$ kips $\boxed{Rext_{DC} = 246.65}$ kips

$\boxed{Rint_{DW} = 27.32}$ kips $\boxed{Rext_{DW} = 27.32}$ kips

E13-1.2.2 Live Load Reactions per Design Lane

Unfactored Live Load Reactions, kips

$$LLR := \begin{pmatrix} \text{"LoadType"} & \text{"Abut"} & \text{"Pier"} \\ \text{"Vehicle"} & 64.72 & 114.17 \\ \text{"Lane"} & 32.76 & 89.41 \end{pmatrix}$$

These loads are per design lane and do not include dynamic load allowance. The pier reactions are controlled by the 90% (Truck Pair + Lane) loading condition. The reactions shown include the 90% factor.

E13-1.3 Select Preliminary Pier Dimensions

Selecting the most optimal pier type depends on site conditions, cost considerations, superstructure geometry, and aesthetics. For this design example, a single column (hammerhead) pier was chosen.

Since the LRFD Specifications do not have standards regarding maximum or minimum dimensions for a pier cap, column, or footing, the designer should base the preliminary pier dimensions on WisDOT specific standards, previous designs, and past experience. The pier cap, however, must be wide enough to accommodate the bearings.

Figures E13-1.3-1 and E13-1.3-2 show the preliminary dimensions selected for this pier design example.

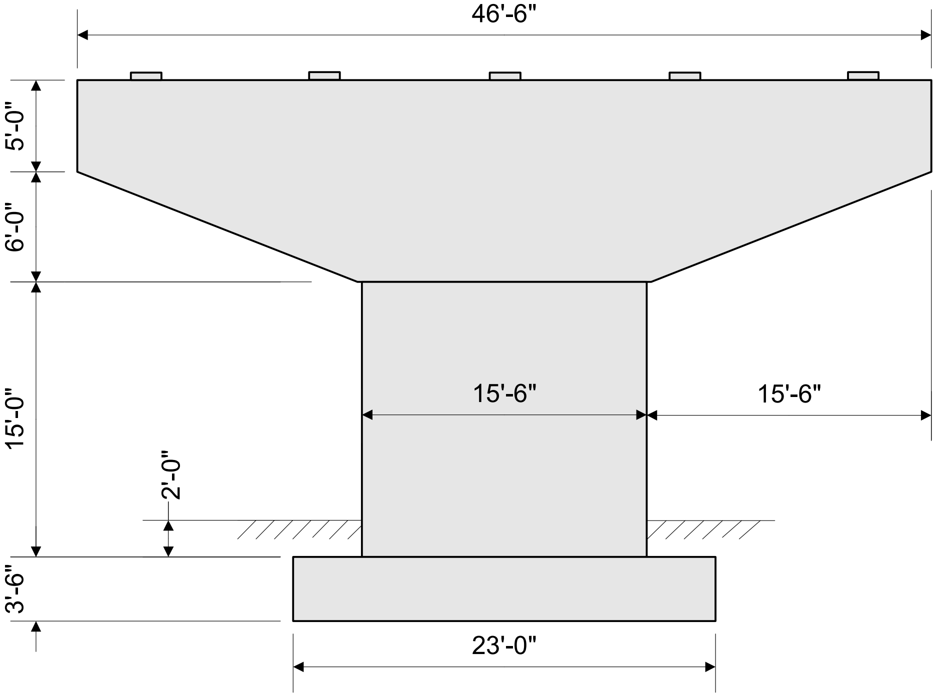


Figure E13-1.3-1
Preliminary Pier Dimensions - Front Elevation

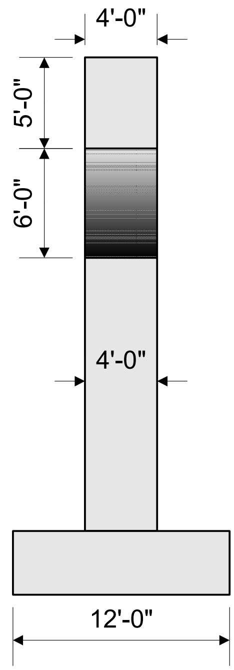


Figure E13-1.3-2
Preliminary Pier Dimensions - End Elevation

Pier Geometry Definitions (feet):

$L_{cap} := 46.5$	$L_{col} := 15.5$	$L_{ftg} := 23$	
$W_{cap} := 4$	$W_{col} := 4$	$W_{ftg} := 12$	
$H_{cap} := 11$	$H_{col} := 15$	$H_{ftg} := 3.5$	
$H_{cap\_end} := 5$		$D_{soil} := 2$	Soil depth above footing, feet
$L_{oh} := 15.5$		$\gamma_{soil} := 0.120$	Unit weight of soil, kcf

E13-1.4 Compute Dead Load Effects

Once the preliminary pier dimensions are selected, the corresponding dead loads can be computed in accordance with **LRFD [3.5.1]**. The pier dead loads must then be combined with the superstructure dead loads.

Exterior girder dead load reactions (DC and DW): $\boxed{Rext_{DC} = 246.65}$ kips

$\boxed{Rext_{DW} = 27.32}$ kips

Interior girder dead load reactions (DC and DW): $\boxed{Rint_{DC} = 241.72}$ kips

$\boxed{Rint_{DW} = 27.32}$ kips

Pier cap dead load:

$$DL_{Cap} := w_c \cdot W_{cap} \cdot \left[2 \cdot \left(\frac{H_{cap\_end} + H_{cap}}{2} \right) \cdot L_{oh} + H_{cap} \cdot L_{col} \right]$$

$$= 0.150 \cdot 4 \cdot \left(2 \cdot \frac{5 + 11}{2} \cdot 15.5 + 11 \cdot 15.5 \right)$$ $\boxed{DL_{Cap} = 251.1}$ kips

Pier column dead load:

$$DL_{col} := w_c \cdot W_{col} \cdot H_{col} \cdot L_{col}$$

$$= 0.150 \cdot 4 \cdot 15 \cdot 15.5$$ $\boxed{DL_{col} = 139.5}$ kips

Pier footing dead load:

$$DL_{ftg} := w_c \cdot W_{ftg} \cdot H_{ftg} \cdot L_{ftg}$$

$$= 0.150 \cdot 12 \cdot 3.5 \cdot 23$$ $\boxed{DL_{ftg} = 144.9}$ kips

In addition to the above dead loads, the weight of the soil on top of the footing must be computed. The two-foot height of soil above the footing was previously defined. Assuming a unit weight of soil at 0.120 kcf in accordance with **LRFD [Table 3.5.1-1]** :

$$EV_{ftg} := \gamma_{soil} \cdot D_{soil} \cdot \left(W_{ftg} \cdot L_{ftg} - W_{col} \cdot L_{col} \right)$$

$$= 0.120 \cdot 2 \cdot (12 \cdot 23 - 4 \cdot 15.5)$$ $\boxed{EV_{ftg} = 51.36}$ kips

E13-1.5 Compute Live Load Effects

For the pier in this design example, the maximum live load effects in the pier cap, column and footing are based on either one, two or three lanes loaded (whichever results in the worst force effect). Figure E13-1.5-1 illustrates the lane positions when three lanes are loaded.

The positioning shown in Figure E13-1.5-1 is determined in accordance with **LRFD [3.6.1].** The first step is to calculate the number of design lanes, which is the integer part of the ratio of the clear roadway width divided by 12 feet per lane. Then the lane loading, which occupies ten feet of the lane, and the HL-93 truck loading, which has a six-foot wheel spacing and a two-foot clearance to the edge of the lane, are positioned within each lane to maximize the force effects in each of the respective pier components.

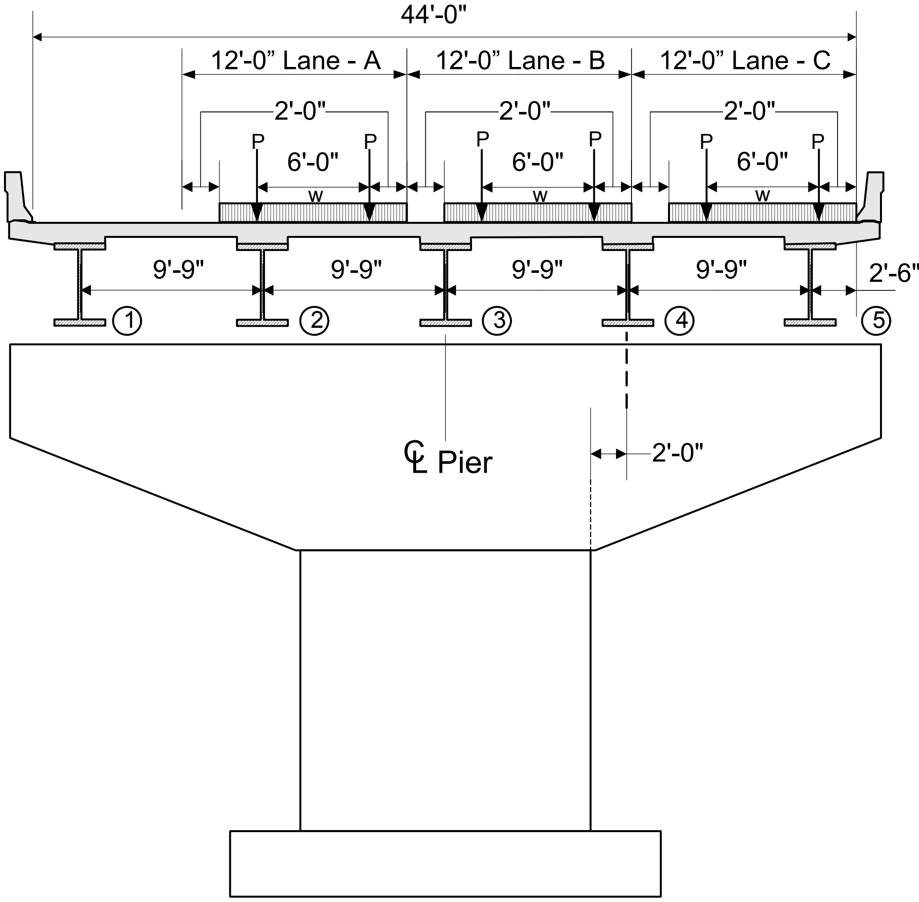


Figure E13-1.5-1
Pier Live Loading

N = maximum number of design lanes that the bridge can accommodate
$w_{roadway}$ = roadway width between curbs, ignoring any median strip
W = design lane width

$W := 12$ feet

$\boxed{w_{roadway} = 44}$ feet

$$N := \frac{w_{roadway}}{W}$$ $\boxed{N = 3.67}$

$\boxed{N = 3}$ design lanes

The unfactored girder reactions for lane load and truck load are obtained from the superstructure analysis and are as shown in E13-1.1.3.2. These reactions do not include dynamic load allowance and are given on a per lane basis (i.e., distribution factor = 1.0). Also, the reactions include the ten percent reduction permitted by the Specifications for interior pier reactions that result from longitudinally loading the superstructure with a truck pair in conjunction with lane loading **LRFD [3.6.1.3.1]**.

Live load reactions at Pier (w/o distribution):

$R_{truck} = 114.17$ kips

$R_{lane} = 89.41$ kips

$IM := 0.33$ Dynamic load allowance, IM from **LRFD [Table 3.6.2.1-1]**

The values of the unfactored concentrated loads which represent the girder truck pair load reaction per wheel line in Figure E13-1.5-1 are:

$$P_{wheel} := \frac{R_{truck}}{2} \cdot (1 + IM)$$ $P_{wheel} = 75.92$ kips

The value of the unfactored uniformly distributed load which represents the girder lane load reaction in Figure E13-1.5-1 is computed next. This load is transversely distributed over ten feet and is not subject to dynamic load allowance, **LRFD [3.6.2.1]**.

$$W_{lane} := \frac{R_{lane}}{10}$$ $W_{lane} = 8.94$ $\frac{kips}{ft}$

The next step is to compute the reactions due to the above loads at each of the five bearing locations. This is generally carried out by assuming the deck is pinned (i.e., discontinuous) at the interior girder locations but continuous over the exterior girders. Solving for the reactions is then elementary. The computations for the reactions with only Lane C loaded are illustrated below as an example. The subscripts indicate the bearing location and the lane loaded to obtain the respective reaction:

$$R_{5\_c} := \frac{P_{wheel} \cdot (4.25 + 10.25) + W_{lane} \cdot 10 \times 7.25}{9.75}$$ $R_{5\_c} = 179.4$ kips

$$R_{4\_c} := P_{wheel} \cdot 2 + W_{lane} \cdot 10 - R_{5\_c}$$ $R_{4\_c} = 61.86$ kips

The reactions at bearings 1, 2 and 3 with only Lane C loaded are zero. Calculations similar to those above yield the following live load reactions with the remaining lanes loaded. All reactions shown are in kips.

Lane A Loaded	Lane B Loaded	Lane C Loaded
$R_{5\_a} := 0.0$	$R_{5\_b} := 0.0$	$R_{5\_c} = 179.4$
$R_{4\_a} := 0.0$	$R_{4\_b} = 123.66$	$R_{4\_c} = 61.86$
$R_{3\_a} = 72.31$	$R_{3\_b} = 117.56$	$R_{3\_c} := 0.0$
$R_{2\_a} = 164.67$	$R_{2\_b} := 0.0$	$R_{2\_c} := 0.0$
$R_{1\_a} = 4.27$	$R_{1\_b} := 0.0$	$R_{1\_c} := 0.0$

E13-1.6 Compute Other Load Effects

Other load effects that will be considered for this pier design include braking force, wind loads, and temperature loads.

For simplicity, buoyancy, stream pressure, ice loads and earthquake loads are not included in this design example.

E13-1.6.1 Braking Force

Since expansion bearings exist at the abutments, the entire longitudinal braking force is resisted by the pier.

In accordance with **LRFD [3.6.4]**, the braking force per lane is the greater of:

25 percent of the axle weights of the design truck or tandem

5 percent of the axle weights of the design truck plus lane load

5 percent of the axle weights of the design tandem plus lane load

The total braking force is computed based on the number of design lanes in the same direction. It is assumed in this example that this bridge is likely to become one-directional in the future. Therefore, any and all design lanes may be used to compute the governing braking force. Also, braking forces are not increased for dynamic load allowance in accordance with **LRFD [3.6.2.1]**. The calculation of the braking force for a single traffic lane follows:

25 percent of the design truck:

$BRK_{trk} := 0.25 \cdot (32 + 32 + 8)$ $\boxed{BRK_{trk} = 18}$ kips

25 percent of the design tandem:

$BRK_{tan} := 0.25 \cdot (25 + 25)$ $\boxed{BRK_{tan} = 12.5}$ kips

5 percent of the axle weights of the design truck plus lane load:

$BRK_{trk\_lan} := 0.05 \cdot [(32 + 32 + 8) + (0.64 \times 2 \cdot L)]$ $\boxed{BRK_{trk\_lan} = 11.28}$ kips

5 percent of the axle weights of the design tandem plus lane load:

$BRK_{tan\_lan} := 0.05 \cdot [(25 + 25) + (0.64 \times 2 \cdot L)]$ $\boxed{BRK_{tan\_lan} = 10.18}$ kips

Use:

$BRK := \max(BRK_{trk}, BRK_{tan}, BRK_{trk\_lan}, BRK_{tan\_lan})$ $\boxed{BRK = 18}$ kips per lane

LRFD [3.6.4] states that the braking force is applied along the longitudinal axis of the bridge at a distance of six feet above the roadway surface. However, since the skew angle is zero for this design example and the bearings are assumed incapable of transmitting longitudinal moment, the braking force will be applied at the top of bearing elevation. For bridges with skews, the component of the braking force in the transverse direction would be applied six feet above the roadway surface.

This force may be applied in either horizontal direction (back or ahead station) to cause the maximum force effects. Additionally, the total braking force is typically assumed equally distributed among the bearings:

$$BRK_{brg} := \frac{BRK}{5} \qquad \boxed{BRK_{brg} = 3.6}$$ kips per bearing per lane

The moment arm about the base of the column is:

$$H_{BRK} := H_{col} + H_{cap} + \frac{H_{brng}}{12} \qquad \boxed{H_{BRK} = 26.53}$$ feet

E13-1.6.2 Wind Load from Superstructure

Prior to calculating the wind load on the superstructure, the structure must be checked for aeroelastic instability, **LRFD [3.8.3]**. If the span length to width or depth ratio is greater than 30, the structure is considered wind-sensitive and design wind loads should be based on wind tunnel studies. This wind load applies to Strength III, Strength V, and Service I.

$\boxed{H_{par} = 2.67}$ Parapet height, feet

Span Length (L): $\boxed{L = 120}$ feet

$Width := w_{deck}$ $\boxed{Width = 46.5}$ feet

$Depth := H_{super} - H_{par}$ $\boxed{Depth = 5.79}$ feet

$\boxed{\frac{L}{Width} = 2.58}$ < 30 OK $\boxed{\frac{L}{Depth} = 20.72}$ < 30 OK

Since the span length to width and depth ratios are both less than 30, the structure does not need to be investigated for aeroelastic instability.

To compute the wind load on the superstructure, the area of the superstructure exposed to the wind must be defined. For this example, the exposed area is the total superstructure depth, (H_{super}), multiplied by length tributary to the pier. Due to expansion bearings at the abutment, the transverse length tributary to the pier is not the same as the longitudinal length.

The superstructure depth includes the total depth from the top of the barrier to the bottom of the girder. Included in this depth is any haunch and/or depth due to the deck cross-slope. Once the total depth is known, (H_{super}), the exposed wind area can be calculated and the design wind pressure applied.

The total depth was previously computed in Section E13-1.1 and is as follows:
$\boxed{H_{super} = 8.46}$ feet

For this two-span bridge example, the tributary length for wind load on the fixed pier in the transverse direction is one-half of each adjacent span:

$$L_{windT} := \frac{L + L}{2} \qquad \boxed{L_{windT} = 120} \quad \text{feet}$$

In the longitudinal direction, the tributary length is the entire bridge length due to the expansion bearings at the abutments:

$$L_{windL} := L \cdot 2 \qquad \boxed{L_{windL} = 240} \quad \text{feet}$$

The transverse wind area is:

$$A_{wsuperT} := H_{super} \cdot L_{windT} \qquad \boxed{A_{wsuperT} = 1015} \quad \text{ft}^2$$

The longitudinal wind area is:

$$A_{wsuperL} := H_{super} \cdot L_{windL} \qquad \boxed{A_{wsuperL} = 2031} \quad \text{ft}^2$$

The design wind pressures applied to the superstructure are shown in Section 13.4.4. To calculate the wind pressure to be used for Strength III, the value of (Z) must be calculated to select the value of (K_Z) in **LRFD [Table C3.8.1.2.1-1]**.

The value of (Z) at the pier is:

$$Z_{pier} := H_{col} - D_{soil} + H_{cap} + H_{super} + \frac{H_{brng}}{12} \qquad \boxed{Z_{pier} = 32.99} \quad \text{feet}$$

Therefore, the average value of (Z) will be less than 33 feet, and because the Wind Exposure Category C applies to this structure, use:

$K_Z := 1.0$; therefore $Psup_{III} = 0.044$ ksf

Because the maximum height above low ground or water level to top of structure is (Z_{pier}), which is 33 feet, and individual span lengths are less than 150 feet, the values for transverse and longitudinal wind forces may be calculated using the simplified method in Section 13.4.4.1.

Strength III:

$Psup_{transIII} := 0.044$ ksf (transverse)

$Psup_{longitIII} := 0.011$ ksf (longitudinal)

Strength V:

$Psup_{transV} := 0.021$ ksf (transverse)

$Psup_{longitV} := 0.006$ ksf (longitudinal)

Service I:

$Psup_{transI} := 0.016$ ksf (transverse)

$Psup_{longitI} := 0.004$ ksf (longitudinal)

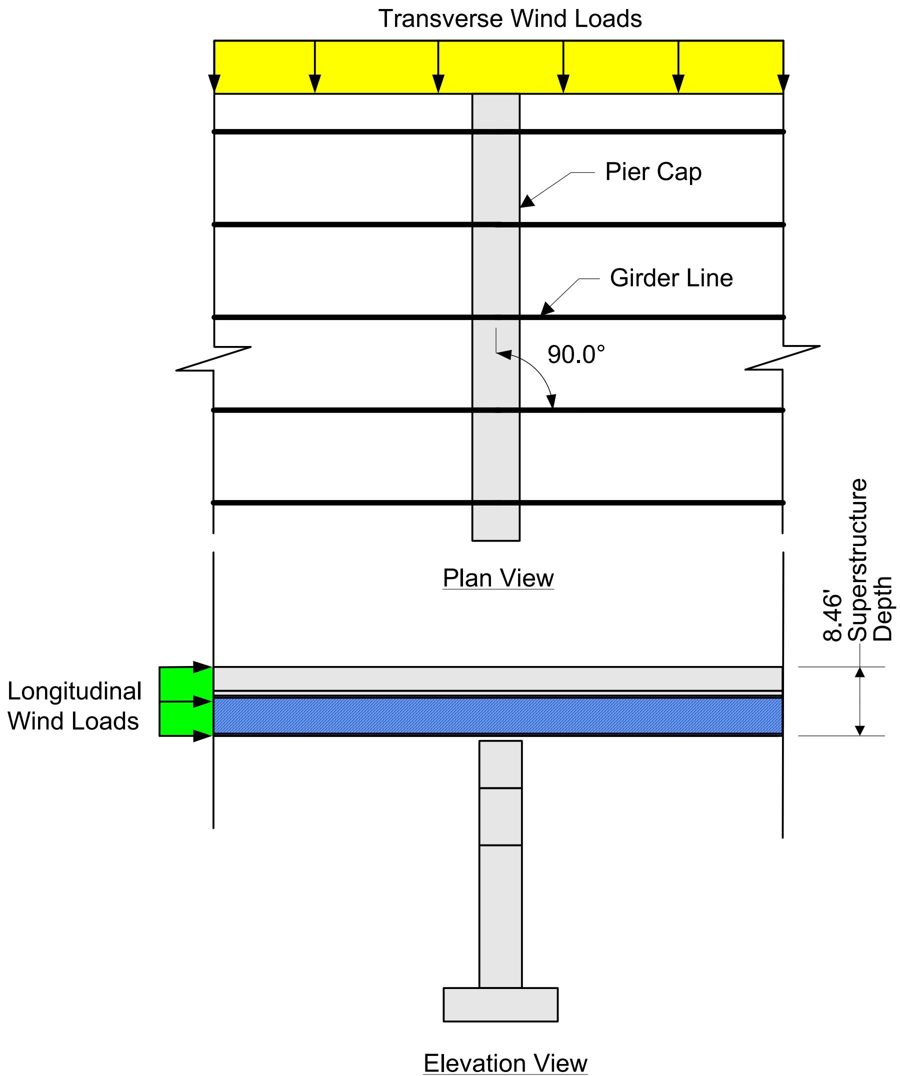


Figure E13-1.6-1

Application of Wind Load

The transvers and longitudinal superstructure wind loads acting on the pier (girders) are:

Strength III:

$WS_{suptrnsIII} := A_{wsuperT} \cdot Psup_{transIII}$ $\boxed{WS_{suptrnsIII} = 44.68}$ kips

$WS_{suplngIII} := A_{wsuperL} \cdot Psup_{longitIII}$ $\boxed{WS_{suplngIII} = 22.34}$ kips

Strength V:

$WS_{suptrnsV} := A_{wsuperT} \cdot Psup_{transV}$ $\boxed{WS_{suptrnsV} = 21.32}$ kips

$WS_{suplngV} := A_{wsuperL} \cdot Psup_{longitV}$ $\boxed{WS_{suplngV} = 12.18}$ kips

Service I:

$WS_{suptrnsI} := A_{wsuperT} \cdot Psup_{transI}$ $\boxed{WS_{suptrnsI} = 16.25}$ kips

$WS_{suplngI} := A_{wsuperL} \cdot Psup_{longitI}$ $\boxed{WS_{suplngI} = 8.12}$ kips

The total longitudinal wind loads (WS_{suplng}) shown above is assumed to be divided equally among the bearings. In addition, the load at each bearing is assumed to be applied at the top of the bearing. These assumptions are consistent with those used in determining the bearing forces due to the longitudinal braking force.

The horizontal force (WS_L) at each bearing due to the longitudinal wind loads on the superstructure are:

$WS_{L\_III} := \frac{WS_{suplngIII}}{5}$ $\boxed{WS_{L\_III} = 4.47}$ kips

$WS_{L\_V} := \frac{WS_{suplngV}}{5}$ $\boxed{WS_{L\_V} = 2.44}$ kips

$WS_{L\_I} := \frac{WS_{suplngI}}{5}$ $\boxed{WS_{L\_I} = 1.62}$ kips

The transverse wind loads ($WS_{suptrns}$) shown above are also assumed to be equally divided among the bearings but are applied at the mid-depth of the superstructure.

The horizontal force (WS_T) at each bearing due to the transverse wind loads on the superstructure are:

$$WS_{T\_III} := \frac{WS_{suptrnsIII}}{5}$$ $\boxed{WS_{T\_III} = 8.94}$ kips

$$WS_{T\_V} := \frac{WS_{suptrnsV}}{5}$$ $\boxed{WS_{T\_V} = 4.26}$ kips

$$WS_{T\_I} := \frac{WS_{suptrnsI}}{5}$$ $\boxed{WS_{T\_I} = 3.25}$ kips

These horizontal forces (WS_T) are shown in Figure E13-1.6-2

For calculating the resulting moment effect on the column, the moment arm about the base of the column for transverse and longitudinal wind forces are:

$$H_{WSlong} := H_{col} + H_{cap} + \frac{H_{brng}}{12}$$ $\boxed{H_{WSlong} = 26.53}$ feet

$$H_{WStrns} := H_{col} + H_{cap} + \frac{H_{brng}}{12} + \frac{H_{super}}{2}$$ $\boxed{H_{WStrns} = 30.76}$ feet

However, the transverse load also applies a moment to the pier cap. This moment, which acts about the centerline of the pier cap, induces vertical loads at the bearings as illustrated in Figure E13-1.6-2. The computations for these vertical forces are presented below.

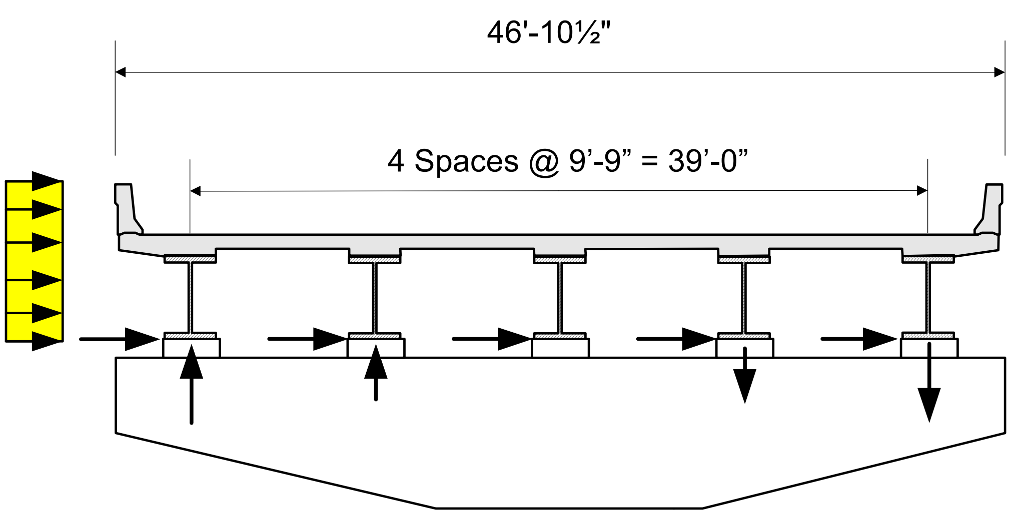


Figure E13-1.6-2
Transverse Wind Loads at Pier Bearings from Wind on Superstructure

Transverse Moments on the Pier Cap:

$M_{trnsIII} := WS_{suptrnsIII} \cdot \left(\frac{H_{super}}{2}\right)$ $\boxed{M_{trnsIII} = 189.02}$ kip-ft

$M_{trnsV} := WS_{suptrnsV} \cdot \left(\frac{H_{super}}{2}\right)$ $\boxed{M_{trnsV} = 90.22}$ kip-ft

$M_{trnsI} := WS_{suptrnsI} \cdot \left(\frac{H_{super}}{2}\right)$ $\boxed{M_{trnsI} = 68.74}$ kip-ft

Moment of Inertia for the Girder Group:

$I = \Sigma A \cdot y^2$

$A = 1 \quad I_1 = I_5 \quad I_2 = I_4 \quad I_3 = 0$ $\boxed{S = 9.75}$ feet (girder spacing)

$I_{girders} := 2 \cdot (S + S)^2 + 2 \cdot S^2$

$= 2 \cdot (9.75 + 9.75)^2 + 2 \cdot 9.75^2$ $\boxed{I_{girders} = 950.63}$ ft^2

$\text{Reaction} = \frac{\text{Moment} \cdot y}{I}$

Vertical Forces at the Bearings:

$RWS1\_5_{trnsIII} := \frac{M_{trnsIII} \cdot (S + S)}{I_{girders}}$ $\boxed{RWS1\_5_{trnsIII} = 3.88}$ kips

$RWS1\_5_{trnsV} := \frac{M_{trnsV} \cdot (S + S)}{I_{girders}}$ $\boxed{RWS1\_5_{trnsV} = 1.85}$ kips

$RWS1\_5_{trnsI} := \frac{M_{trnsI} \cdot (S + S)}{I_{girders}}$ $\boxed{RWS1\_5_{trnsI} = 1.41}$ kips

The loads at bearings 1 and 5 are equal but opposite in direction. Similarly for bearings 2 and 4:

$RWS2\_4_{trnsIII} := \frac{M_{trnsIII} \cdot S}{I_{girders}}$ $\boxed{RWS2\_4_{trnsIII} = 1.94}$ kips

$$RWS2\_4_{trnsV} := \frac{M_{trnsV} \cdot S}{I_{girders}}$$ $\boxed{RWS2\_4_{trnsV} = 0.93}$ kips

$$RWS2\_4_{trnsl} := \frac{M_{trnsl} \cdot S}{I_{girders}}$$ $\boxed{RWS2\_4_{trnsl} = 0.70}$ kips

Finally, by inspection: $\boxed{RWS3_{trns} = 0}$ kips

These vertical forces (RWS) are shown in Figure E13-1.6-2

E13-1.6.2.1 Vertical Wind Load

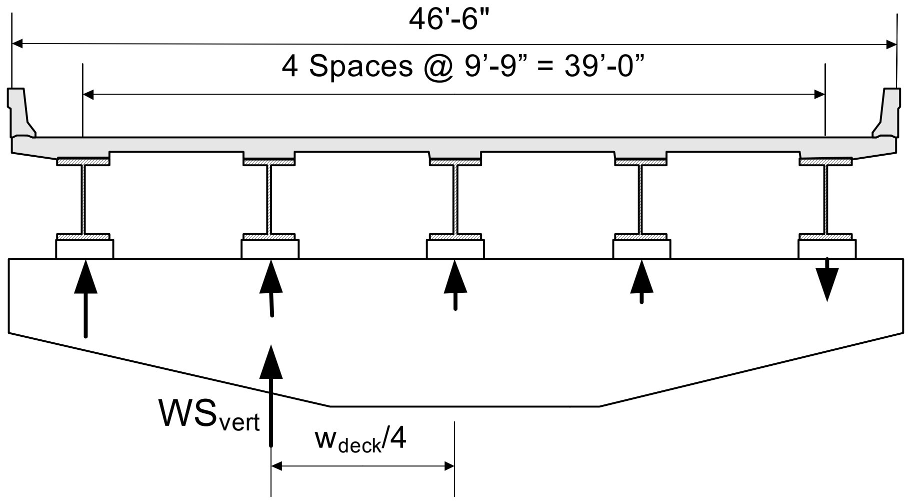
The vertical (upward) wind load is calculated by multiplying a <u>0.020 ksf</u> vertical wind pressure by the out-to-out bridge deck width as described in Section 13.4.4.4. It is applied at the windward quarter-point of the deck only for limit states that do not include wind on live load (Strength III). The wind load is then multiplied by the tributary length, which is one-half of each adjacent span.

From previous definitions:

$\boxed{w_{deck} = 46.5}$ ft $\boxed{L_{windT} = 120}$ ft

The total vertical wind load is then:

$$WS_{vert} := 0.02(w_{deck}) \cdot (L_{windT})$$ $\boxed{WS_{vert} = 111.6}$ kips



<u>Figure E13-1.6-3</u>
Vertical Wind Loads at Pier Bearings from Wind on Superstructure

This load causes a moment about the pier centerline. The value of this moment is:

$M_{WS\_vert} := WS_{vert} \cdot \frac{w_{deck}}{4}$ $\boxed{M_{WS\_vert} = 1297.35}$ kip-ft

The vertical loads at the bearings are computed as: $\boxed{S = 9.75}$ feet (girder spacing)

$RWS_{vert1} := \frac{-WS_{vert}}{5} + \frac{M_{WS\_vert} \cdot (2 \cdot S)}{I_{girders}}$ $\boxed{RWS_{vert1} = 4.29}$ kips

$RWS_{vert2} := \frac{-WS_{vert}}{5} + \frac{M_{WS\_vert} \cdot S}{I_{girders}}$ $\boxed{RWS_{vert2} = -9.01}$ kips

$RWS_{vert3} := \frac{-WS_{vert}}{5}$ $\boxed{RWS_{vert3} = -22.32}$ kips

$RWS_{vert4} := \frac{-WS_{vert}}{5} - \frac{M_{WS\_vert} \cdot S}{I_{girders}}$ $\boxed{RWS_{vert4} = -35.63}$ kips

$RWS_{vert5} := \frac{-WS_{vert}}{5} - \frac{M_{WS\_vert} \cdot 2 \cdot S}{I_{girders}}$ $\boxed{RWS_{vert5} = -48.93}$ kips

Where a negative value indicates a vertical upward load. These loads only apply to Strength III.

E13-1.6.2.2 Wind Load on Vehicles

The representation of wind pressure acting on vehicular traffic is given by **LRFD [3.8.1.3]** as a uniformly distributed line load. This load is applied both transversely and longitudinally. For the transverse and longitudinal loadings, the total force in each respective direction is calculated by multiplying the appropriate component by the length of structure tributary to the pier. Similar to the superstructure wind loading, the longitudinal length tributary to the pier differs from the transverse length. As shown in E13-1.6.2, the criteria for using the simplified method in Section 13.4.4.3 has been met, and the transverse and longitudinal loads are calculated as shown below and are to be applied simultaneously. This wind load applies to Strength V and Service I.

$\boxed{L_{windT} = 120}$ feet $\boxed{L_{windL} = 240}$ feet

$P_{LLtrans} := 0.100$ klf

$P_{LLlongit} := 0.040$ klf

$WL_{trans} := L_{windT} \cdot P_{LLtrans}$ $\boxed{WL_{trans} = 12}$ kips

$WL_{long} := L_{windL} \cdot P_{LLlongit}$ $\boxed{WL_{long} = 9.6}$ kips

The wind on vehicular live loads shown above are applied to the bearings in the same manner as the wind load from the superstructure. That is, the total transverse and longitudinal load is equally distributed to each bearing and applied at the top of the bearing.

The horizontal forces (WL_T , WL_L) at each bearing due to wind load on vehicles are:

$$WL_{T\_V} := \frac{WL_{trans}}{5}$$ $\boxed{WL_{T\_V} = 2.4}$ kips

$$WL_{T\_I} := \frac{WL_{trans}}{5}$$ $\boxed{WL_{T\_I} = 2.4}$ kips

$$WL_{L\_V} := \frac{WL_{long}}{5}$$ $\boxed{WL_{L\_V} = 1.92}$ kips

$$WL_{L\_I} := \frac{WL_{long}}{5}$$ $\boxed{WL_{L\_I} = 1.92}$ kips

In addition, the transverse load acting six feet above the roadway applies a moment to the pier cap. This moment induces vertical reactions at the bearings. The values of these vertical reactions are given below. The computations for these reactions are not shown but are carried out as shown in E13-1.6.2. The only difference is that the moment arm used for calculating the moment is equal to (H_{super} - H_{par} + 6.0 feet).

$$Mom_{arm} := H_{super} - H_{par} + 6$$ $\boxed{Mom_{arm} = 11.79}$ feet

Vertical Forces at the Bearings:

$\boxed{RWL1\_5_{trns} = 2.9}$ kips

$\boxed{RWL2\_4_{trns} = 1.45}$ kips

$\boxed{RWL3_{trns} = 0}$ kips

For calculating the resulting moment effect on the column, the moment arm about the base of the column is:

$$H_{WLlong} := H_{col} + H_{cap} + \frac{H_{brng}}{12}$$ $\boxed{H_{WLlong} = 26.53}$ feet

$$H_{WLtrns} := H_{col} + H_{cap} + \frac{H_{brng}}{12} + \left(H_{super} - H_{par} + 6\right)$$ $\boxed{H_{WLtrns} = 38.32}$ feet

E13-1.6.3 Wind Load on Substructure

The design wind pressure applied directly to the substructure units are shown in Section 13.4.4. As stated in E13-1.6.2, for Strength III the value of K_Z =1.0. For simplicity, apply the same pressure in the transverse and longitudinal directions for Strength III, V and Service I.

Strength III:

$P_{subIII} := 0.054$ ksf (transverse/longitudinal)

Strength V:

$P_{subV} := 0.026$ ksf (transverse/longitudinal)

Service I:

$P_{subI} := 0.020$ ksf (transverse/longitudinal)

In accordance with Section 13.4.4.2, the transverse and longitudinal wind forces calculated from these wind pressures acting on the corresponding exposed areas are to be applied simultaneously. These loads shall also act simultaneously with the superstructure wind loads.

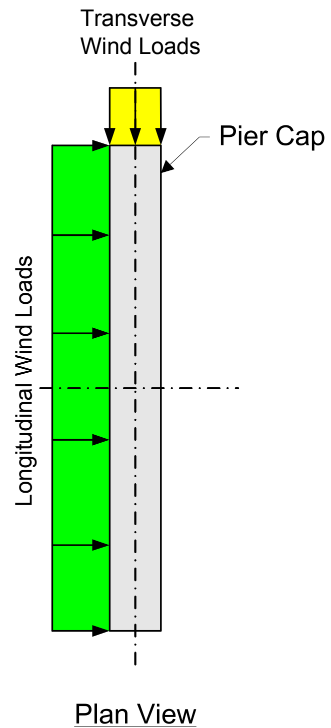


Figure E13-1.6-4
Wind Pressure on Pier

What follows is an example of the calculation of the wind loads acting directly on the pier. For simplicity, the tapers of the pier cap overhangs will be considered solid. The column height exposed to wind is the distance from the ground line (which is two feet above the footing) to the bottom of the pier cap.

Component areas of the pier cap:

$$A_{capLong} := (L_{cap}) \cdot (H_{cap})$$ $\boxed{A_{capLong} = 511.5}$ ft^2

$$A_{capTrans} := (W_{cap}) \cdot (H_{cap})$$ $\boxed{A_{capTrans} = 44}$ ft^2

Component areas of the pier column:

$$A_{colLong} := (L_{col}) \cdot (H_{col} - D_{soil})$$ $\boxed{A_{colLong} = 201.5}$ ft^2

$$A_{colTrans} := (W_{col}) \cdot (H_{col} - D_{soil})$$ $\boxed{A_{colTrans} = 52}$ ft^2

The transverse and longitudinal substructure wind loads acting on the pier are:

Strength III:

$$WS_{subLIII} := P_{subIII} \cdot (A_{capLong} + A_{colLong})$$ $\boxed{WS_{subLIII} = 38.5}$ kips

$$WS_{subTIII} := P_{subIII} \cdot (A_{capTrans} + A_{colTrans})$$ $\boxed{WS_{subTIII} = 5.18}$ kips

Strength V:

$$WS_{subLV} := P_{subV} \cdot (A_{capLong} + A_{colLong})$$ $\boxed{WS_{subLV} = 18.54}$ kips

$$WS_{subTV} := P_{subV} \cdot (A_{capTrans} + A_{colTrans})$$ $\boxed{WS_{subTV} = 2.50}$ kips

Service I:

$$WS_{subLI} := P_{subI} \cdot (A_{capLong} + A_{colLong})$$ $\boxed{WS_{subLI} = 14.26}$ kips

$$WS_{subTI} := P_{subI} \cdot (A_{capTrans} + A_{colTrans})$$ $\boxed{WS_{subTI} = 1.92}$ kips

The point of application of these loads will be the centroid of the loaded area of each face, respectively.

$$H_{WSsubL} := \frac{A_{capLong} \cdot \left(H_{col} + \frac{H_{cap}}{2}\right) + A_{colLong} \cdot \left(\frac{H_{col} - 2}{2} + 2\right)}{A_{capLong} + A_{colLong}}$$

$\boxed{H_{WSsubL} = 17.11}$ feet

$$H_{WSsubT} := \frac{A_{capTrans} \cdot \left(H_{col} + \frac{H_{cap}}{2}\right) + A_{colTrans} \cdot \left(\frac{H_{col} - 2}{2} + 2\right)}{A_{capTrans} + A_{colTrans}}$$

$\boxed{H_{WSsubT} = 14}$ feet

E13-1.6.4 Temperature Loading (Superimposed Deformations)

In this particular structure, with a single pier centered between two abutments that have identical bearing types, the temperature force is based on assuming a minimum coefficient of expansion at one abutment and the maximum at the other using only dead load reactions. This force acts in the longitudinal direction of the bridge (either back or ahead station) and is equally divided among the bearings. Also, the forces at each bearing from this load will be applied at the top of the bearing.

The abutment girder Dead Load reactions from E13-1.2.1 are as follows:

$\boxed{AbutRint_{DC} = 61.69}$ $\boxed{AbutRext_{DC} = 62.97}$

$\boxed{AbutRint_{DW} = 7.46}$ $\boxed{AbutRext_{DW} = 7.46}$

$\boxed{\mu_{min} = 0.06}$

$\boxed{\mu_{max} = 0.1}$

$\Delta\mu := \mu_{max} - \mu_{min}$ $\boxed{\Delta\mu = 0.04}$

$$F_{TU} := \Delta\mu \cdot \left[3 \cdot \left(AbutRint_{DC} + AbutRint_{DW}\right) + 2 \cdot \left(AbutRext_{DC} + AbutRext_{DW}\right)\right]$$

$\boxed{F_{TU} = 13.93}$ kips

The resulting temperature force acting on each bearing is:

$TU_{BRG} := \frac{F_{TU}}{5}$ $\boxed{TU_{BRG} = 2.79}$ kips

The moment arm about the base of the column is:

$H_{TU} := H_{col} + H_{cap} + \frac{H_{brng}}{12}$ $\boxed{H_{TU} = 26.53}$ feet

E13-1.7 Analyze and Combine Force Effects

The first step within this design step will be to summarize the loads acting on the pier at the bearing locations. This is done in Tables E13-1.7-1 through E13-1.7-8 shown below. Tables E13-1.7-1 through E13-1.7-5 summarize the vertical loads, Tables E13-1.7-6 through E13-1.7-7 summarize the horizontal longitudinal loads, and Table E13-1.7-8 summarizes the horizontal transverse loads. These loads along with the pier self-weight loads, which are shown after the tables, need to be factored and combined to obtain total design forces to be resisted in the pier cap, column and footing.

It will be noted here that loads applied due to braking and temperature can act either ahead or back station. Also, wind loads can act on either side of the structure and with positive or negative skew angles. This must be kept in mind when considering the signs of the forces in the tables below. The tables assume a particular direction for illustration only.

	Superstructure Dead Load		Wearing Surface Dead Load	
Bearing	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)
1	$Rext_{DC}$	246.65	$Rext_{DW}$	27.32
2	$Rint_{DC}$	241.72	$Rint_{DW}$	27.32
3	$Rint_{DC}$	241.72	$Rint_{DW}$	27.32
4	$Rint_{DC}$	241.72	$Rint_{DW}$	27.32
5	$Rext_{DC}$	246.65	$Rext_{DW}$	27.32

Table E13-1.7-1
Unfactored Vertical Bearing Reactions from Superstructure Dead Load

	Vehicular Live Load **					
	Lane A		Lane B		Lane C	
Bearing	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)
1	R_1_a	4.27	R_1_b	0.00	R_1_c	0.00
2	R_2_a	164.67	R_2_b	0.00	R_2_c	0.00
3	R_3_a	72.31	R_3_b	117.56	R_3_c	0.00
4	R_4_a	0.00	R_4_b	123.66	R_4_c	61.86
5	R_5_a	0.00	R_5_b	0.00	R_5_c	179.40

**Note: Live load reactions include impact on truck loading.

Table E13-1.7-2
Unfactored Vertical Bearing Reactions from Live Load

Strength III

Bearing No.	Reactions from Transverse Wind Load on Superstructure (kips)
1	3.88
2	1.94
3	0.00
4	-1.94
5	-3.88

Strength V

Bearing No.	Reactions from Transverse Wind Load on Superstructure (kips)
1	1.85
2	0.93
3	0.00
4	-0.93
5	-1.85

Service I

Bearing No.	Reactions from Transverse Wind Load on Superstructure (kips)
1	1.41
2	0.70
3	0.00
4	-0.70
5	-1.41

Table E13-1.7-3
Unfactored Vertical Bearing Reactions from Wind on Superstructure

Strength V , Service I

Bearing No.	Reactions from Transverse Wind Load on Vehicular Live Load (kips)
1	2.90
2	1.45
3	0.00
4	-1.45
5	-2.90

Table E13-1.7-4
Unfactored Vertical Bearing Reactions from Wind on Live Load

Strength III

Bearing No.	Vertical Wind Load on Superstructure: Variable Name	Reaction (Kips)
1	RWS_{vert1}	4.29
2	RWS_{vert2}	-9.01
3	RWS_{vert3}	-22.32
4	RWS_{vert4}	-35.63
5	RWS_{vert5}	-48.93

Table E13-1.7-5
Unfactored Vertical Bearing Reactions from Vertical Wind on Superstructure

	Braking Load **		Temperature Loading	
	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)
Each Bearing	BRK_{brg}	3.60	TU_{BRG}	2.79

**Note: Values shown are for a single lane loaded

Table E13-1.7-6
Unfactored Horizontal Longitudinal Bearing Reactions from Braking and Temperature

Strength III

Load Type	Unfactored Horizontal Longitudinal Forces (kips)
Wind Loads from Superstructure	22.34
Wind on Live Load	0.00
Wind on Pier	38.50

Strength V

Load Type	Unfactored Horizontal Longitudinal Forces (kips)
Wind Loads from Superstructure	12.18
Wind on Live Load	9.60
Wind on Pier	18.54

Service I

Load Type	Unfactored Horizontal Longitudinal Forces (kips)
Wind Loads from Superstructure	8.12
Wind on Live Load	9.60
Wind on Pier	14.26

Table E13-1.7-7
Unfactored Horizontal Longitudinal Forces

Strength III

Load Type	Unfactored Horizontal Transverse Forces (kips)
Wind Loads from Superstructure	44.68
Wind on Live Load	0.00
Wind on Pier	5.18

Strength V

Load Type	Unfactored Horizontal Transverse Forces (kips)
Wind Loads from Superstructure	21.32
Wind on Live Load	12.00
Wind on Pier	2.50

Service I

Load Type	Unfactored Horizontal Transverse Forces (kips)
Wind Loads from Superstructure	16.25
Wind on Live Load	12.00
Wind on Pier	1.92

Table E13-1.7-8
Unfactored Horizontal Transverse Forces

In addition to all the loads tabulated above, the pier self-weight must be considered when determining the final design forces. Additionally for the footing and pile designs, the weight of the earth on top of the footing must be considered. These loads were previously calculated and are shown below:

$\boxed{DL_{Cap} = 251.1}$ kips $\boxed{DL_{ftg} = 144.9}$ kips

$\boxed{DL_{col} = 139.5}$ kips $\boxed{EV_{ftg} = 51.36}$ kips

In the AASHTO LRFD design philosophy, the applied loads are factored by statistically calibrated load factors. In addition to these factors, one must be aware of two additional sets of factors which may further modify the applied loads.

The first set of additional factors applies to all force effects and are represented by the Greek letter η (eta) in the Specifications, **LRFD [1.3.2.1]**. These factors are related to the ductility, redundancy, and operational importance of the structure. A single, combined eta is required for every structure. In accordance with WisDOT policy, all eta factors are taken equal to one.

The other set of factors mentioned in the first paragraph above applies only to the live load force effects and are dependent upon the number of loaded lanes. These factors are termed multiple presence factors by the Specifications, **LRFD [Table 3.6.1.1.2-1]**. These factors for this bridge are shown as follows:

Multiple presence factor, m (1 lane) $m_1 := 1.20$

Multiple presence factor, m (2 lanes) $m_2 := 1.00$

Multiple presence factor, m (3 lanes) $m_3 := 0.85$

Table E13-1.7-9 contains the applicable limit states and corresponding load factors that will be used for this pier design. Limit states not shown either do not control the design or are not applicable. The load factors shown in Table E13-1.7-9 are the standard load factors assigned by the Specifications and are exclusive of multiple presence and eta factors.

It is important to note here that the maximum load factors shown in Table E13-1.7-9 for uniform temperature loading (TU) apply only for deformations, and the minimum load factors apply for all other effects. Since the force effects from the uniform temperature loading are considered in this pier design, the minimum load factors will be used.

	Load Factors							
	Strength I		Strength III		Strength V		Service I	
Load	γ_{max}	γ_{min}	γ_{max}	γ_{min}	γ_{max}	γ_{min}	γ_{max}	γ_{min}
DC	1.25	0.90	1.25	0.90	1.25	0.90	1.00	1.00
DW	1.50	0.65	1.50	0.65	1.50	0.65	1.00	1.00
LL	1.75	1.75	---	---	1.35	1.35	1.00	1.00
BR	1.75	1.75	---	---	1.35	1.35	1.00	1.00
TU	1.20	0.50	1.20	0.50	1.20	0.50	1.20	1.00
WS	---	---	1.00	1.00	1.00	1.00	1.00	1.00
WL	---	---	---	---	1.00	1.00	1.00	1.00
EV	1.35	1.00	1.35	1.00	1.35	1.00	1.00	1.00

Table E13-1.7-9
Load Factors and Applicable Pier Limit States

The loads discussed and tabulated previously can now be factored by the appropriate load factors and combined to determine the governing limit states in the pier cap, column, footing and piles. Design calculations will be carried out for the governing limit states only.

E13-1.7.1 Pier Cap Force Effects

The pier cap will be designed using a strut and tie model. See E13-1.8 for additional information. For this type of model, the member's self weight is included in the bearing reactions. The calculation of the Strength I Factored girder reactions follows.

For the dead load of the cap, the tributary weight of the cap will be added to each girder reaction.

$$Cap_{DC\_1} := 8.625 \cdot \frac{5 + 8.34}{2} \cdot W_{cap} \cdot w_c \qquad \boxed{Cap_{DC\_1} = 34.52} \quad \text{kips}$$

$$Cap_{DC\_2} := \left(6.875 \cdot \frac{8.34 + 11}{2} + 2.875 \cdot 11\right) \cdot W_{cap} \cdot w_c \qquad \boxed{Cap_{DC\_2} = 58.86} \quad \text{kips}$$

$$Cap_{DC\_3} := 9.75 \cdot 11 \cdot W_{cap} \cdot w_c \qquad \boxed{Cap_{DC\_3} = 64.35} \quad \text{kips}$$

$$Cap_{DC\_4} := Cap_{DC\_2} \qquad \boxed{Cap_{DC\_4} = 58.86} \quad \text{kips}$$

$$Cap_{DC\_5} := Cap_{DC\_1} \qquad \boxed{Cap_{DC\_5} = 34.52} \quad \text{kips}$$

Look at the combined live load girder reactions with 1 (Lane C), 2 (Lanes C and B) and 3 lanes (Lanes C, B and A) loaded. The multiple presence factor from E13-1.7 shall be applied. The design lane locations were located to maximize the forces over the right side of the cap.

	Unfactored Vehicular Live Load					
	1 Lane, m=1.2		2 Lanes, m=1.0		3 Lanes, m=0.85	
Bearing	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)
1	R_1_1	0.00	R_1_2	0.00	R_1_3	3.63
2	R_2_1	0.00	R_2_2	0.00	R_2_3	139.97
3	R_3_1	0.00	R_3_2	117.56	R_3_3	161.40
4	R_4_1	74.23	R_4_2	185.52	R_4_3	157.70
5	R_5_1	215.27	R_5_2	179.40	R_5_3	152.49

Table E13-1.7-10
Unfactored Vehicular Live Load Reactions

Calculate the Strength I Combined Girder Reactions for 1, 2 and 3 lanes loaded. An example calculation is shown for the girder 5 reaction with one lane loaded. Similar calculations are performed for the remaining girders and number of lanes loaded.

$$Ru_{5\_1} := \gamma_{DCmax} \cdot \left(Rext_{DC} + Cap_{DC\_5}\right) + \gamma_{DWmax} \cdot Rext_{DW} + \gamma_{LL} \cdot R_{5\_1}$$

$$\boxed{Ru_{5\_1} = 769.17} \quad \text{kips}$$

	Total Factored Girder Reactions**					
	1 Lane, m=1.2		2 Lanes, m=1.0		3 Lanes, m=0.85	
Bearing	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)
1	Ru_1_1	392.44	Ru_1_2	392.44	Ru_1_3	398.79
2	Ru_2_1	416.71	Ru_2_2	416.71	Ru_2_3	661.66
3	Ru_3_1	423.57	Ru_3_2	629.30	Ru_3_3	706.01
4	Ru_4_1	546.62	Ru_4_2	741.38	Ru_4_3	692.68
5	Ru_5_1	769.17	Ru_5_2	706.38	Ru_5_3	659.29

** Includes dead load of pier cap

Table E13-1.7-11

Factored Girder Reactions for STM Cap Design

E13-1.7.2 Pier Column Force Effects

The controlling limit states for the design of the pier column are Strength V (for biaxial bending with axial load). The critical design location is where the column meets the footing, or at the column base. The governing force effects for Strength V are achieved by minimizing the axial effects while maximizing the transverse and longitudinal moments. This is accomplished by excluding the future wearing surface, applying minimum load factors on the structure dead load, and loading only Lane B and Lane C with live load.

For Strength V, the factored vertical forces and corresponding moments at the critical section are shown below.

Strength V Axial Force:

$\boxed{Rext_{DC} = 246.65}$ kips $\boxed{R_{3\_2} = 117.56}$ kips

$\boxed{Rint_{DC} = 241.72}$ kips $\boxed{R_{4\_2} = 185.52}$ kips

$\boxed{DL_{Cap} = 251.1}$ kips $\boxed{R_{5\_2} = 179.4}$ kips

$\boxed{DL_{col} = 139.5}$ kips

$$Ax_{colStrV} := \gamma_{DCminStrV} \cdot \left(2 \cdot Rext_{DC} + 3 \cdot Rint_{DC} + DL_{Cap} + DL_{col}\right) \ldots$$
$$+ \gamma_{LLStrV}\left(R_{3\_2} + R_{4\_2} + R_{5\_2}\right)$$

$\boxed{Ax_{colStrV} = 2099.51}$ kips

Strength V Transverse Moment:

$\boxed{S = 9.75}$ feet (girder spacing)

$ArmV3_{col} := 0$ $\boxed{ArmV3_{col} = 0}$ feet

$ArmV4_{col} := S$ $\boxed{ArmV4_{col} = 9.75}$ feet

$ArmV5_{col} := 2 \cdot S$ $\boxed{ArmV5_{col} = 19.5}$ feet

$\boxed{WS_{suptrnsV} = 21.32}$ kips $\boxed{H_{WStrns} = 30.76}$ feet

$\boxed{WL_{trans} = 12}$ kips $\boxed{H_{WLtrns} = 38.32}$ feet

$\boxed{WS_{subTV} = 2.5}$ kips $\boxed{H_{WSsubT} = 14}$ feet

$$MuT_{colStrV} := \gamma_{LLStrV}\left(R_{3\_2} \cdot ArmV3_{col} + R_{4\_2} \cdot ArmV4_{col} + R_{5\_2} \cdot ArmV5_{col}\right) \ldots$$
$$+ \gamma_{WLStrV} \cdot \left(WL_{trans} \cdot H_{WLtrns}\right) \ldots$$
$$+ \gamma_{WSStrV} \cdot \left(WS_{suptrnsV} \cdot H_{WStrns} + WS_{subTV} \cdot H_{WSsubT}\right)$$

$\boxed{MuT_{colStrV} = 8315.32}$ kip-ft

Strength V Longitudinal Moment:

$\boxed{BRK_{brg} = 3.6}$ kips/bearing per lane $\boxed{H_{BRK} = 26.53}$ feet

$\boxed{TU_{BRG} = 2.79}$ kips/ bearing $\boxed{H_{TU} = 26.53}$ feet

$\boxed{WS_{suplngV} = 12.18}$ kips $\boxed{H_{WSlong} = 26.53}$ feet

$\boxed{WL_{long} = 9.6}$ kips $\boxed{H_{WLlong} = 26.53}$ feet

$\boxed{WS_{subLV} = 18.54}$ kips $\boxed{H_{WSsubL} = 17.11}$ feet

$\boxed{m_2 = 1.00}$ multi presence factor for two lanes loaded

$$MuL_{colStrV} := \gamma_{BRStrV} \cdot \left(5 \cdot BRK_{brg} \cdot H_{BRK} \cdot 2 \cdot m_2\right) \ldots$$
$$+ \gamma_{TUminStrV}\left(5TU_{BRG} \cdot H_{TU}\right) \ldots$$
$$+ \gamma_{WLStrV} \cdot \left(WL_{long} \cdot H_{WLlong}\right) \ldots$$
$$+ \gamma_{WSStrV} \cdot \left(WS_{suplngV} \cdot H_{WSlong} + WS_{subLV} \cdot H_{WSsubL}\right)$$

$\boxed{MuL_{colStrV} = 2369.38}$ kip-ft

For Strength III, the factored transverse shear in the column is:

$\boxed{WS_{subTIII} = 5.18}$ kips $\boxed{WS_{suptrnsIII} = 44.68}$ kips

$VuT_{col} := \gamma_{WSStrIII}\left(WS_{suptrnsIII} + WS_{subTIII}\right)$ $\boxed{VuT_{col} = 49.86}$ kips

For Strength V, the factored longitudinal shear in the column is (reference Table E13-1.7-7):

$\boxed{WL_{long} = 9.6}$ kips $\boxed{WS_{subLV} = 18.54}$ kips $\boxed{WS_{suplngV} = 12.18}$ kips

$$VuL_{col} := \gamma_{WSStrV}\left(WS_{suplngV} + WS_{subLV}\right) + \gamma_{WLStrV} \cdot WL_{long} \cdots$$
$$+ \gamma_{TUmin} \cdot \left(TU_{BRG} \cdot 5\right) + \gamma_{BRStrV} \cdot \left(5 \cdot BRK_{brg}\right) \cdot 3 \cdot m_3$$

$\boxed{VuL_{col} = 109.25}$ kips

E13-1.7.3 Pier Pile Force Effects

The foundation system for the pier is a reinforced concrete footing on steel H-piles. The force effects in the piles cannot be determined without a pile layout. The pile layout depends upon the pile capacity and affects the footing design. The pile layout used for this pier foundation is shown in Figure E13-1.10-1.

Based on the pile layout shown in Figure E13-1.10-1, the controlling limit states for the pile design are Strength I (for maximum pile load), Strength V (for minimum pile load), and Strength III and Strength V (for maximum horizontal loading of the pile group).

Structure Dead Load Effects:

Girder DC Reactions:

$\boxed{Rext_{DC} = 246.65}$ kips

$\boxed{Rint_{DC} = 241.72}$ kips

Girder DW Reactions:

$\boxed{Rext_{DW} = 27.32}$ kips

$\boxed{Rint_{DW} = 27.32}$ kips

$DC_{Super} := 2 \cdot Rext_{DC} + 3 \cdot Rint_{DC}$ $\boxed{DC_{Super} = 1218.46}$ kips

$\boxed{DL_{Cap} = 251.1}$ kips

$\boxed{DL_{col} = 139.5}$ kips

$\boxed{DL_{ftg} = 144.9}$ kips

$DC_{pile} := DC_{Super} + DL_{Cap} + DL_{col} + DL_{ftg}$ $\boxed{DC_{pile} = 1753.96}$ kips

$DW_{pile} := 2 \cdot Rext_{DW} + 3 \cdot Rint_{DW}$ $\boxed{DW_{pile} = 136.6}$ kips

Vertical Earth Load Effects:

$EV_{pile} := EV_{ftg}$ $\boxed{EV_{pile} = 51.36}$ kips

Live Load Effects (without Dynamic Load Allowance)

Live Load Girder Reactions for 2 lanes, Lanes B and C, loaded:

$\boxed{R_{1\_2p} = 0}$ kips

$\boxed{R_{2\_2p} = 0}$ kips

$\boxed{R_{3\_2p} = 99.21}$ kips

$\boxed{R_{4\_2p} = 156.54}$ kips

$\boxed{R_{5\_2p} = 151.38}$ kips $\boxed{R_{T\_2p} = 407.13}$ kips

From Section E13-1.7, the Transverse moment arm for girders 3, 4 and 5 are:

$\boxed{ArmV3_{col} = 0}$ feet

$\boxed{ArmV4_{col} = 9.75}$ feet

$\boxed{ArmV5_{col} = 19.5}$ feet

The resulting Transverse moment applied to the piles is:

$$M_{LL2T\_p} := R_{3\_2p} \cdot ArmV3_{col} + R_{4\_2p} \cdot ArmV4_{col} + R_{5\_2p} \cdot ArmV5_{col}$$

$\boxed{M_{LL2T\_p} = 4478.2}$ kip-ft

The Longitudinal Strength I Moment includes the braking and temperature forces.

$$MuL2_{colStr1} := \gamma_{BR} \cdot (5 \cdot BRK_{brg} \cdot H_{BRK} \cdot 2 \cdot m_2) + \gamma_{TUmin}(5TU_{BRG} \cdot H_{TU})$$

$\boxed{MuL2_{colStr1} = 1856.29}$ kip-ft

Strength I Load for Maximum Pile Reaction

The maximum pile load results from the Strength I load combination with two lanes loaded.

$$Pu2_{pile\_Str1} := \gamma_{DCmax} \cdot DC_{pile} + \gamma_{DWmax} \cdot DW_{pile} + \gamma_{EVmax} \cdot EV_{pile} + \gamma_{LL} \cdot R_{T\_2p}$$

$\boxed{Pu2_{pile\_Str1} = 3179.17}$ kips

$MuT2_{pile\_Str1} := \gamma_{LL} \cdot M_{LL2T\_p}$ $\boxed{MuT2_{pile\_Str1} = 7836.85}$ kip-ft

$MuL2_{pile\_Str1} := MuL2_{colStr1}$ $\boxed{MuL2_{pile\_Str1} = 1856.29}$ kip-ft

Minimum Load on Piles Strength V

The calculation for the minimum axial load on piles is similar to the Strength V axial column load calculated previously. The weight of the footing and soil surcharge are included. The girder reactions used for pile design do not include impact. The DW loads are not included.

$$Pu_{pile\_StrV} := \gamma_{DCminStrV} \cdot \left(2 \cdot Rext_{DC} + 3 \cdot Rint_{DC} + DL_{Cap} + DL_{col} + DL_{ftg}\right) \dots + \gamma_{EVminStrV} \cdot EV_{pile} \dots + \gamma_{LLStrV}\left(R_{3\_2p} + R_{4\_2p} + R_{5\_2p}\right)$$

$\boxed{Pu_{pile\_StrV} = 2179.55}$ kips

The calculation for the Strength V longitudinal moment is the same as the longitudinal moment on the column calculated previously. These loads include the braking force, temperature, wind on live load and wind on the structure.

$$MuL_{pile\_StrV} := \gamma_{BRStrV} \cdot \left(5 \cdot BRK_{brg} \cdot H_{BRK} \cdot 2 \cdot m_2\right) \dots + \gamma_{TUminStrV}\left(5TU_{BRG} \cdot H_{TU}\right) \dots + \gamma_{WLStrV} \cdot \left(WL_{long} \cdot H_{WLlong}\right) \dots + \gamma_{WSStrV} \cdot \left(WS_{suplngV} \cdot H_{WSlong} + WS_{subLV} \cdot H_{WSsubL}\right)$$

$\boxed{MuL_{pile\_StrV} = 2369.38}$ kip-ft

The calculation for the Strength V transverse moment is the similar as the transverse moment on the column calculated previously. These loads include the live load, wind on live load and wind on the structure. Impact is not included in these live load reactions.

$$MuT_{pile\_StrV} := \gamma_{LLStrV}\left(R_{3\_2p} \cdot ArmV3_{col} + R_{4\_2p} \cdot ArmV4_{col} + R_{5\_2p} \cdot ArmV5_{col}\right) \dots + \gamma_{WLStrV} \cdot \left(WL_{trans} \cdot H_{WLtrns}\right) \dots + \gamma_{WSStrV} \cdot \left(WS_{suptrnsV} \cdot H_{WStrns} + WS_{subTV} \cdot H_{WSsubT}\right)$$

$\boxed{MuT_{pile\_StrV} = 7196.34}$ kip-ft

For Strength III, the factored transverse shear in the footing is equal to the transverse force at the base of the column.

$$HuT_{pileStrIII} := VuT_{col}$$

$$= \gamma_{WSStrIII} \cdot \left(WS_{suptrnsIII} + WS_{subTIII}\right) \qquad \boxed{HuT_{pileStrIII} = 49.86} \text{ kips}$$

For Strength V, the factored longitudinal shear in the column is equal to the longitudinal force at the base of the column.

$$HuL_{pileStrV} := VuL_{col} \qquad \boxed{HuL_{pileStrV} = 109.25} \text{ kips}$$

The following is a summary of the controlling forces on the piles:

Strength I

$\boxed{Pu2_{pile\_Str1} = 3179.17}$ kips

$\boxed{MuT2_{pile\_Str1} = 7836.85}$ kip-ft

$\boxed{MuL2_{pile\_Str1} = 1856.29}$ kip-ft

Strength III

$\boxed{HuT_{pileStrIII} = 49.86}$ kips

Strength V

$\boxed{Pu_{pile\_StrV} = 2179.55}$ kips

$\boxed{MuT_{pile\_StrV} = 7196.34}$ kip-ft

$\boxed{MuL_{pile\_StrV} = 2369.38}$ kip-ft

$\boxed{HuL_{pileStrV} = 109.25}$ kips

E13-1.7.4 Pier Footing Force Effects

The controlling limit states for the design of the pier footing are Strength I (for flexure, punching shear at the column, and punching shear at the maximum loaded pile, and for one-way shear). In accordance with Section 13.11, the footings do not require the crack control by distribution check in **LRFD [5.7.3.4]**. As a result, the Service I Limit State is not required. There is not a single critical design location in the footing where all of the force effects just mentioned are checked. Rather, the force effects act at different locations in the footing and must be checked at their respective locations. For example, the punching shear checks are carried out using critical perimeters around the column and maximum loaded pile, while the flexure and one-way shear checks are carried out on a vertical face of the footing either parallel or perpendicular to the bridge longitudinal axis. Also note that impact is not included for members that are below ground. The weight of the footing concrete and the soil above the footing are not included in these loads as they counteract the pile reactions.

$DC_{ftg} := DC_{Super} + DL_{Cap} + DL_{col}$ $\boxed{DC_{ftg} = 1609.06}$ kips

$DW_{ftg} := 2 \cdot Rext_{DW} + 3 \cdot Rint_{DW}$ $\boxed{DW_{ftg} = 136.6}$ kips

Unfactored Live Load reactions for one, two and three lanes loaded:

$\boxed{R_{T\_1p} = 244.3}$ kips

$\boxed{R_{T\_2p} = 407.13}$ kips

$\boxed{R_{T\_3p} = 519.1}$ kips

The resulting Transverse moment applied to the piles is:

$M_{LL1T} := R_{4\_1p} \cdot ArmV4_{col} + R_{5\_1p} \cdot ArmV5_{col}$ $\boxed{M_{LL1T} = 4153.03}$ kip-ft

$M_{LL2T} := R_{4\_2p} \cdot ArmV4_{col} + R_{5\_2p} \cdot ArmV5_{col}$ $\boxed{M_{LL2T} = 4478.2}$ kip-ft

$M_{LL3T} := \left(-R_{2\_3p} + R_{4\_3p}\right) \cdot ArmV4_{col} + \left(-R_{1\_3p} + R_{5\_3p}\right) \cdot ArmV5_{col}$

$\boxed{M_{LL3T} = 2595.17}$ kip-ft

The maximum pile load results from the Strength I load combination with two lanes loaded.

$Pu2_{ftgStr1} := \gamma_{DCmax} \cdot DC_{ftg} + \gamma_{DWmax} \cdot DW_{ftg} + \gamma_{LL} \cdot R_{T\_2p}$

$\boxed{Pu2_{ftgStr1} = 2928.7}$ kips

$MuT2_{ftgStr1} := \gamma_{LL} \cdot M_{LL2T}$ $\boxed{MuT2_{ftgStr1} = 7836.85}$ kip-ft

$MuL2_{ftgStr1} := \gamma_{BR} \cdot \left(5 \cdot BRK_{brg} \cdot H_{BRK} \cdot 2 \cdot m_2\right) \ldots + \gamma_{TUmin}\left(5TU_{BRG} \cdot H_{TU}\right)$ $\boxed{MuL2_{ftgStr1} = 1856.29}$ kip-ft

The Strength I limit state controls for the punching shear check at the column. In this case the future wearing surface is included, maximum factors are applied to all the dead load components, and all three lanes are loaded with live load. This results in the following bottom of column forces:

$Pu3_{ftgStr1} := \gamma_{DCmax} \cdot DC_{ftg} + \gamma_{DWmax} \cdot DW_{ftg} + \gamma_{LL} \cdot R_{T\_3p}$

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

$MuT3_{ftgStr1} := \gamma_{LL} \cdot M_{LL3T}$ $\boxed{MuT3_{ftgStr1} = 4541.55}$ kip-ft

$MuL3_{ftgStr1} := \gamma_{BR} \cdot (5 \cdot BRK_{brg} \cdot H_{BRK} \cdot 3 \cdot m_3) \ldots + \gamma_{TUmin}(5TU_{BRG} \cdot H_{TU})$ $\boxed{MuL3_{ftgStr1} = 2315.94}$ kip-ft

E13-1.8 Design Pier Cap - Strut and Tie Method (STM)

Prior to carrying out the actual design of the pier cap, a brief discussion is in order regarding the design philosophy that will be used for the design of the structural components of this pier.

When a structural member meets the definition of a deep component **LRFD [5.6.3.1]**, the LRFD Specifications recommend, although it does not mandate, that the strut-and-tie method be used to determine force effects and required reinforcing. **LRFD [C5.6.3.1]** indicates that a strut-and-tie model properly accounts for nonlinear strain distribution, nonuniform shear distribution, and the mechanical interaction of V_u, T_u and M_u. Use of strut-and-tie models for the design of reinforced concrete members is new to the LRFD Specification. WisDOT policy is to design hammerhead pier caps using STM.

E13-1.8.1 Determine Geometry and Member Forces

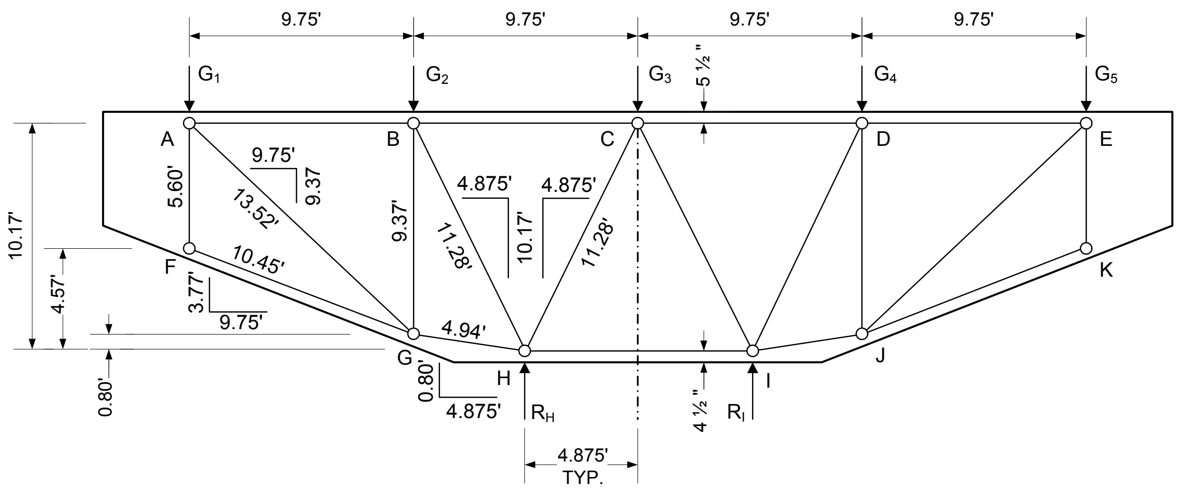


Figure E13-1.8-1
Strut and Tie Model Dimensions

In order to maintain a minimum 25° angle between struts and ties, the support Nodes (H and I) are located midway between the girder reactions **LRFD [5.6.3.2]**. For this example a compressive strut depth of 8 inches will be used, making the centroids of the bottom truss chords 4.5 inches from the concrete surface. It is also assumed that two layers of rebar will be required along the top tension ties, and the centroid is located 5.5 inches below the top of the cap.

$centroid_{bot} := 4.5$ inches $centroid_{top} := 5.5$ inches

Strength I Loads:

	Total Factored Girder Reactions**					
	1 Lane, m=1.2		2 Lanes, m=1.0		3 Lanes, m=0.85	
Bearing	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)
1	Ru_1_1	392.44	Ru_1_2	392.44	Ru_1_3	398.79
2	Ru_2_1	416.71	Ru_2_2	416.71	Ru_2_3	661.66
3	Ru_3_1	423.57	Ru_3_2	629.30	Ru_3_3	706.01
4	Ru_4_1	546.62	Ru_4_2	741.38	Ru_4_3	692.68
5	Ru_5_1	769.17	Ru_5_2	706.38	Ru_5_3	659.29

** Includes dead load of pier cap

Table E13-1.8-1
Total Factored Girder Reactions

Calculate the forces in the members for the Strength I Load Case with 2 lanes loaded.

To find the column reaction at Node I, sum moments about Node H:

$$R_{I\_2} := \frac{Ru_{3\_2} \cdot 4.875 + Ru_{4\_2} \cdot 14.625 + Ru_{5\_2} \cdot 24.375 - Ru_{2\_2} \cdot 4.875 - Ru_{1\_2} \cdot 14.625}{9.75}$$

$$\boxed{R_{I\_2} = 2395.66} \quad \text{kips}$$

$$R_{H\_2} := Ru_{1\_2} + Ru_{2\_2} + Ru_{3\_2} + Ru_{4\_2} + Ru_{5\_2} - R_{I\_2}$$

$$\boxed{R_{H\_2} = 490.55} \quad \text{kips}$$

The method of joints is used to calculate the member forces. Start at Node K.

By inspection, the following are zero force members and can be ignored in the model:

$F_{JK} := 0$ $\quad F_{EK} := 0$ $\quad F_{AF} := 0$ $\quad F_{FG} := 0$ kips

Note: All forces shown are in kips. "C" indicates compression and "T" indicates tension.

At Node E:

$$F_{EJ\_vert} := Ru_{5\_2} \qquad \boxed{F_{EJ\_vert} = 706.38}$$

$F_{EJ\_horiz} = Ru_{5\_2} \cdot \frac{EJ_h}{EJ_v}$ $\boxed{F_{EJ\_horiz} = 735.42}$

$F_{EJ} := \sqrt{F_{EJ\_vert}^{\ 2} + F_{EJ\_horiz}^{\ 2}}$ $\boxed{F_{EJ} = 1019.71}$ C

$F_{DE} := F_{EJ\_horiz}$ $\boxed{F_{DE} = 735.42}$ T

At Node J:

$F_{IJ\_horiz} := F_{EJ\_horiz}$ $\boxed{F_{IJ\_horiz} = 735.42}$

$F_{IJ\_vert} = F_{IJ\_horiz} \cdot \frac{0.802}{4.875}$ $\boxed{F_{IJ\_vert} = 120.99}$

$F_{IJ} := \sqrt{F_{IJ\_horiz}^{\ 2} + F_{IJ\_vert}^{\ 2}}$ $\boxed{F_{IJ} = 745.31}$ C

$F_{DJ} := F_{EJ\_vert} - F_{IJ\_vert}$ $\boxed{F_{DJ} = 585.4}$ T

At Node D:

$F_{DI\_vert} := F_{DJ} + Ru_{4\_2}$ $\boxed{F_{DI\_vert} = 1326.77}$

$F_{DI\_horiz} = F_{DI\_vert} \cdot \frac{4.875}{10.167}$ $\boxed{F_{DI\_horiz} = 636.18}$

$F_{DI} := \sqrt{F_{DI\_vert}^{\ 2} + F_{DI\_horiz}^{\ 2}}$ $\boxed{F_{DI} = 1471.41}$ C

$F_{CD} := F_{DE} + F_{DI\_horiz}$ $\boxed{F_{CD} = 1371.6}$ T

At Node I:

$\boxed{R_{I\_2} = 2395.66}$

$F_{CI\_vert} := R_{I\_2} - F_{DI\_vert} - F_{IJ\_vert}$ $\boxed{F_{CI\_vert} = 947.9}$

$F_{CI\_horiz} = F_{CI\_vert} \cdot \frac{4.875}{10.167}$ $\boxed{F_{CI\_horiz} = 454.51}$

$F_{CI} := \sqrt{F_{CI\_vert}^{\ 2} + F_{CI\_horiz}^{\ 2}}$ $\boxed{F_{CI} = 1051.23}$ C

$F_{HI} := F_{DI\_horiz} + F_{IJ\_horiz} - F_{CI\_horiz}$ $\boxed{F_{HI} = 917.09}$ C

Similar calculations are performed to determine the member forces for the remainder of the model and for the load cases with one and three lanes loaded. The results are summarized in the following figures:

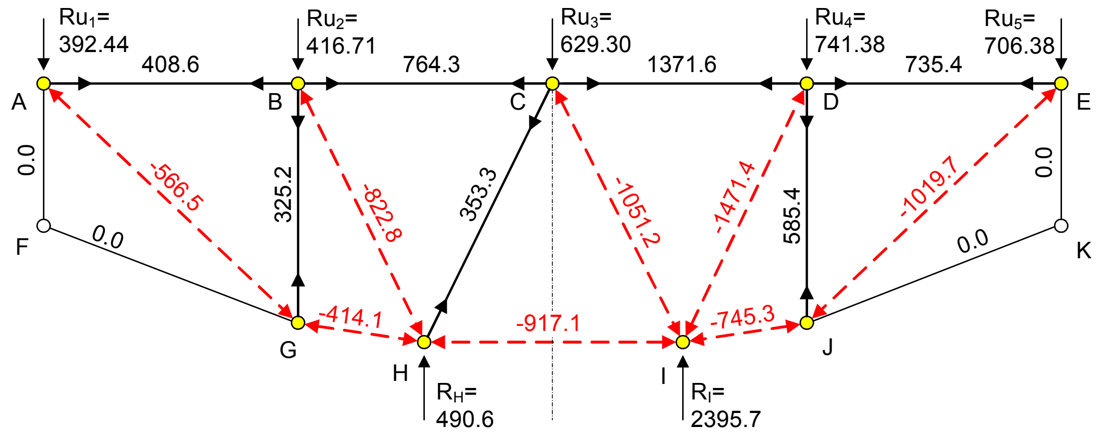


Figure E13-1.8-2
STM Member Forces (Two Lanes Loaded)

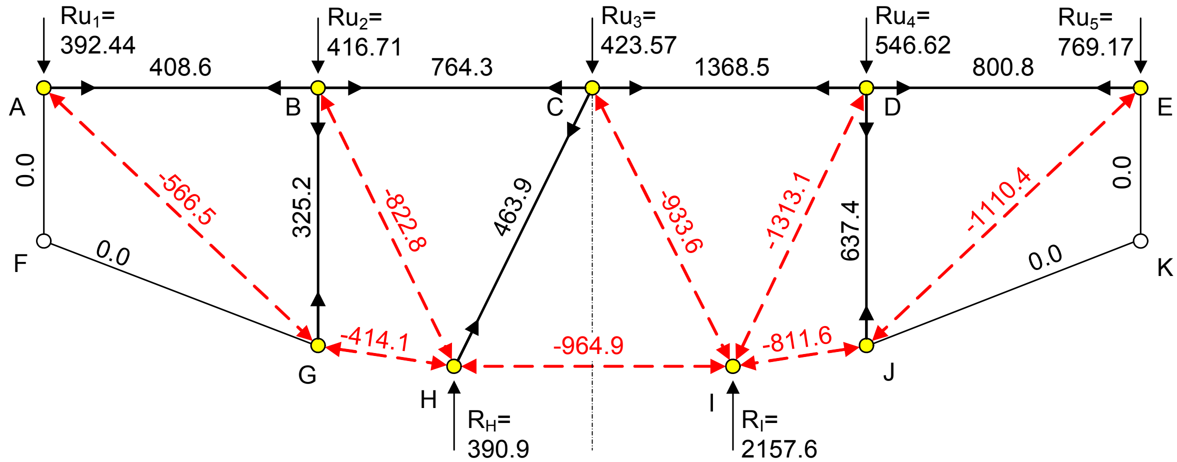


Figure E13-1.8-3
STM Member Forces (One Lane Loaded)

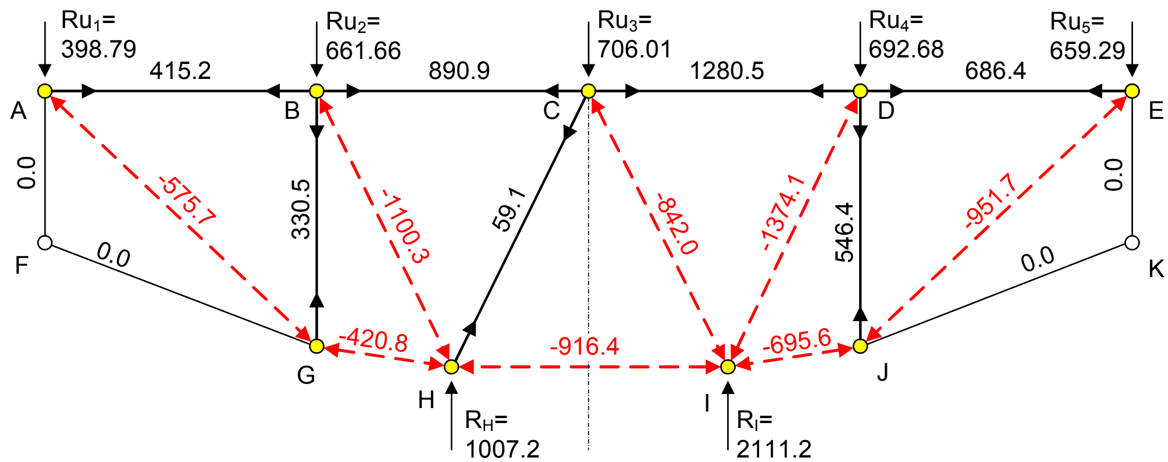


Figure E13-1.8-4
STM Member Forces (Three Lanes Loaded)

E13-1.8.2 Check the Size of the Bearings

The node types are defined by the combinations of struts and ties meeting at the node.

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

The nominal resistance (P_n) at the bearing node face is computed based on the limiting compressive stress (f_{cu}), and the effective area beneath the bearing device ($A_{bearing}$) **LRFD [5.6.3.5]**.

$$P_n = f_{cu} A_{bearing} = (m\, \nu\, f'_c)\, A_{bearing}$$

where:

m = Confinement modification factor **LRFD [5.7.5]**

ν = Concrete efficiency factor **LRFD [5.6.3.5.3a]**

therefore, $A_{bearing} \geq P_u \,/\, \phi_{brg}\,(m\, \nu\, f'_c)$

Bearing Nodes: Nodes A & E: (CCT) Node C: (CTT) Nodes B & D: (CTT)

The nodes located at the bearings are either (CTT) or (CCT) nodes, and the largest loads for these types are present at Nodes D and E respectively. Conservatively use m=1.0, and analyze for crack control reinforcement being present.

At Node D the bearing area required is: (CTT)

$A_{bearing} \geq P_u \ / \ \phi_{brg}\left(m \cdot 0.65 \cdot f'_c\right)$ --- (from Sect. 13.7.3)

At Node E the bearing area required is: (CCT)

$A_{bearing} \geq P_u \ / \ \phi_{brg}\left(m \cdot 0.70 \cdot f'_c\right)$ --- (from Sect. 13.7.3)

$m := 1.0$ $\phi_{brg} := 0.70$ **LRFD [5.5.4.2]** $f'_c = 3.5$ ksi

Calculate bearing area required for Node D:

$Ru_{4\_2} = 741.38$ kips 2-lanes loaded controls (Fig. E13-1.8-2)

$\gamma_{DCmax} \cdot Cap_{DC\_4} = 73.58$ kips pier cap tributary weight below Node D

$$BrgD_2 := \frac{Ru_{4\_2} - \gamma_{DCmax} \cdot Cap_{DC\_4}}{\phi_{brg} \cdot \left(m \cdot 0.65 \cdot f'_c\right)} \qquad BrgD_2 = 419.34 \quad in^2$$

Calculate bearing area required for Node E:

$Ru_{5\_1} = 769.17$ kips 1-lane loaded controls (Fig. E13-1.8-3)

$\gamma_{DCmax} \cdot Cap_{DC\_5} = 43.15$ kips pier cap tributary weight below Node E

$$BrgE_1 := \frac{Ru_{5\_1} - \gamma_{DCmax} \cdot Cap_{DC\_5}}{\phi_{brg} \cdot \left(m \cdot 0.70 \cdot f'_c\right)} \qquad BrgE_1 = 423.34 \quad in^2$$

$BrgArea := \max\left(BrgD_2, BrgE_1\right)$

The area provided by the (26" x 18") bearing plate is:

$A_{bearing} := L_{brng} \cdot W_{brng}$ $A_{bearing} = 468$ in^2

Is $A_{bearing} \geq BrgArea$? check = "OK"

E13-1.8.3 Calculate the Tension Tie Reinforcement

For the top reinforcement in the pier cap, the maximum area of tension tie reinforcement, (A_{st}), is controlled by Tie CD for two lanes loaded (Fig. E13-1.8-2) and is calculated as follows:

LRFD [5.6.3.4.1]

$\boxed{Pu_{CD\_2} = 1371.6}$ kips

$\phi := 0.9$ **LRFD [5.5.4.2]** $f_y = 60$ ksi

$Ast_{CD} := \dfrac{Pu_{CD\_2}}{\phi \cdot f_y}$ $\boxed{Ast_{CD} = 25.4}$ in^2

Therefore, use one row of 9 No.11 bars and one row of 9 No. 10 bars spaced at 5 inches for the top reinforcement.

$As_{No11} := 1.5625$ in^2

$As_{No10} := 1.2656$ in^2

Total area of top reinforcement is:

$As_{CD} := 9 \cdot As_{No11} + 9 \cdot As_{No10}$ $\boxed{As_{CD} = 25.45}$ in^2

Is $As_{CD} \geq Ast_{CD}$? $\boxed{check = \text{"OK"}}$

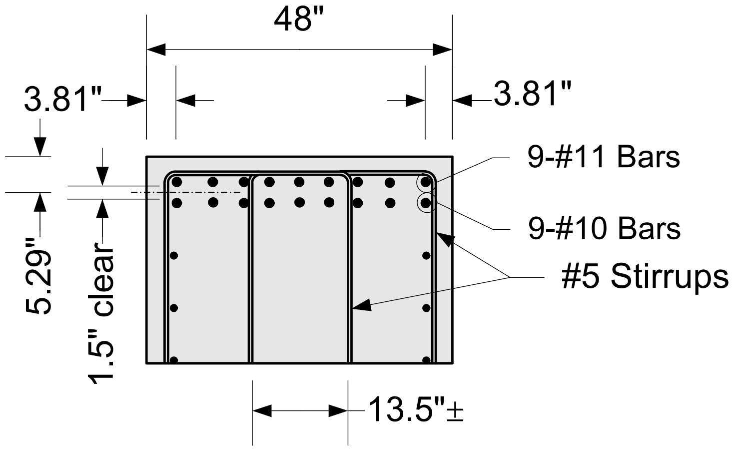


Figure E13-1.8-5
Cap Reinforcement at Tension Tie CD

Note: See **LRFD [5.10.3.1.3]** for spacing requirements between layers of rebar.

For the top reinforcement just inside the exterior girder (Node E), the required area of tension tie reinforcement, (A_{st}), is controlled by Tie DE for one lane loaded (Fig. E13-1.8-3), and is calculated as follows:

$\boxed{Pu_{DE\_1} = 800.79}$ kips

$\phi = 0.9$ **LRFD [5.5.4.2]** $f_y = 60$ ksi

$Ast_{DE} := \frac{Pu_{DE\_1}}{\phi \cdot f_y}$ $\boxed{Ast_{DE} = 14.83}$ in^2

Therefore, use one row of 9 No.11 bars spaced at 5 inches, and one row of 5 No.10 bars for the top reinforcement.

Total area of top reinforcement is:

$As_{DE} := 9 \cdot As_{No11} + 5 \cdot As_{No10}$ $\boxed{As_{DE} = 20.39}$ in^2

Is $As_{DE} \geq Ast_{DE}$? $\boxed{check = \text{"OK"}}$

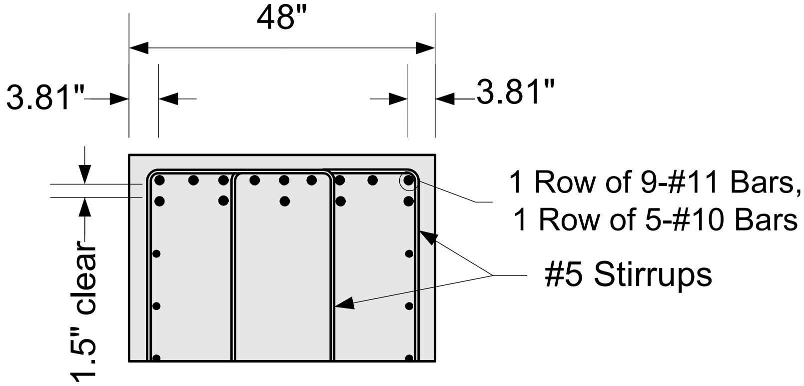


Figure E13-1.8-6
Cap Reinforcement at Tension Tie DE

E13-1.8.4 Calculate the Stirrup Reinforcement

The vertical tension Tie DJ must resist a factored tension force as shown below **LRFD [5.6.3.4.1]**. The controlling force occurs with one lane loaded (Fig. E13-1.8-3). This tension force will be resisted by stirrups within the specified stirrup region length, with the total area of stirrups being (Ast_{DJ}.) Note that any tension ties located directly over the column do not require stirrup design.

$\boxed{Pu_{DJ\_1} = 637.43}$ kips

$\phi = 0.9$ **LRFD [5.5.4.2]** $f_y = 60$ ksi

$Ast_{DJ} := \frac{Pu_{DJ\_1}}{\phi \cdot f_y}$ $\boxed{Ast_{DJ} = 11.8}$ in^2

Try No. 5 bars, with four legs (double-stirrups):

$As_{No5} := 0.3068$ in^2

$Ast := 4 \cdot As_{No5}$ $\boxed{Ast = 1.23}$ in^2

Calculate number of stirrups required:

$n_{DJ} := \frac{Ast_{DJ}}{Ast}$ $n_{DJ} = 9.62$

$\boxed{n_{DJ} = 10}$ bars

The length (L_{DJ}) of the region over which the stirrups shall be distributed for Tie DJ, is from the face of the column to half way between girders 4 and 5 (Nodes D and E).

$\boxed{S = 9.75}$ feet (girder spacing) $\boxed{L_{col} = 15.5}$ feet (column width)

$L_{DJ} := 1.5 \cdot S - \frac{L_{col}}{2}$ $\boxed{L_{DJ} = 6.88}$ feet

Therefore, the required stirrup spacing, s, within this region is:

$s_{stirrup} := \frac{L_{DJ} \cdot 12}{n_{DJ}}$ $\boxed{s_{stirrup} = 8.25}$ in

$\boxed{s_{stirrup} = 8}$ in

Examine stirrups as vertical crack control reinforcement, and their req'd. spacing (s_{cc}) **LRFD [5.6.3.6]**:

$$\frac{Ast}{b_v \cdot s_{cc}} \geq 0.003$$

$b_v := W_{cap} \cdot 12$ $\boxed{b_v = 48}$ in

$s_{cc} := \frac{Ast}{0.003 \cdot b_v}$ $\boxed{s_{cc} = 8.52}$ in

$\boxed{s_{cc} = 8}$ in

$s_{stir} := \min\left(s_{stirrup}, s_{cc}\right)$ $\boxed{s_{stir} = 8}$ in

Therefore, use pairs of (No. 5 bar) double-legged stirrups at 8 inch spacing in the pier cap.

E13-1.8.5 Compression Strut Capacity - Bottom Strut

After the tension tie reinforcement has been designed, the next step is to check the capacity of the compressive struts in the pier cap. Strut IJ carries the highest bottom compressive force when one lane is loaded (Fig. E13-1.8-3). Strut IJ is anchored by Node J, which also anchors Tie DJ and Strut EJ. From the geometry of the idealized internal truss, the smallest angle (α_s) between Tie DJ and Strut IJ is:

$\alpha_s := \text{atan}\left(\frac{IJ_h}{IJ_v}\right)$ $\boxed{\alpha_s = 80.66 \cdot deg}$

$\theta := 90deg - \alpha_s$ $\boxed{\theta = 9.34 \cdot deg}$

$\boxed{Pu_{IJ\_1} = -811.55}$ kips

The nominal resistance (Pn_{IJ}) of Strut IJ is computed based on the limiting compressive stress, (f_{cu}), and the effective cross-sectional area of the strut (Acn_{IJ}) at the node face **LRFD [5.6.3.5]**.

$Pn_{IJ} = f_{cu}\ Acn_{IJ} = (\nu\ f'_c)\ Acn_{IJ}$

where:

ν = Concrete efficiency factor **LRFD [5.6.3.5.3a]**

therefore, $Pu_{IJ\_1} \leq \phi c_{STM} (\nu\ f'_c)\ Acn_{IJ}$

The centroid of the strut was assumed to be at $\boxed{centroid_{bot} = 4.5}$ inches vertically from the bottom face. Therefore at Node J, the thickness of the strut perpendicular to the sloping bottom face (t_{IJ}), and the width (w_{IJ}) of the strut are:

$t_{IJ} := 2 \cdot centroid_{bot} \cdot \cos(\theta)$ $\boxed{t_{IJ} = 8.88}$ inches

$w_{IJ} := W_{cap} \cdot 12$ $\boxed{w_{IJ} = 48}$ inches

$Acn_{IJ} := t_{IJ} \cdot w_{IJ}$ $\boxed{Acn_{IJ} = 426.27}$ in^2

At Node J the node type is (CCT), and the surface where Strut IJ meets the node is a back face. Analyze for crack control reinforcement being present.

At Node J, the capacity of Strut IJ shall satisfy:

$Pu_{IJ\_1} \leq \phi c_{STM} \cdot (0.70 \cdot f'_c) \cdot Acn_{IJ}$ --- (from Sect. 13.7.5)

$\phi c_{STM} := 0.7$ **LRFD [5.5.4.2]** $f'_c = 3.5$ ksi

The factored resistance is:

$$Pr_{IJ} := \phi c_{STM} \cdot (0.70 \cdot f'_c) \cdot Acn_{IJ}$$ $\boxed{Pr_{IJ} = 731.05}$ kips

$\boxed{Pu_{IJ\_1} = 811.55}$ kips

Is $Pr_{IJ} \geq Pu_{IJ\_1}$? $\boxed{check = \text{"No Good"}}$

Because Node J is an interior node not bounded by a bearing plate, it is a smeared node, and a check of concrete strength as shown above is not necessary **LRFD [5.6.3.2]**.

E13-1.8.6 Compression Strut Capacity - Diagonal Strut

Strut DI carries the highest diagonal compressive force when two lanes are loaded (Fig. E13-1.8-2). Strut DI is anchored by Node D, which also anchors Ties CD, DE and DJ. From the geometry of the idealized internal truss, the smallest angle between Ties CD and DE and Strut DI is:

$$\alpha_s := \operatorname{atan}\left(\frac{DI_v}{DI_h}\right)$$ $\boxed{\alpha_s = 64.38 \cdot deg}$

$$\theta := 90deg - \alpha_s$$ $\boxed{\theta = 25.62 \cdot deg}$

$\boxed{Pu_{DI\_2} = -1471.41}$ kips

The cross sectional dimension of Strut DI in the plane of the pier at Node D is calculated as follows. Note that for skewed bearings, the length of the bearing is the projected length along the centerline of the pier cap.

$\boxed{L_{brng} = 26}$ inches $\boxed{W_{brng} = 18}$ inches

$\boxed{centroid_{top} = 5.5}$ inches

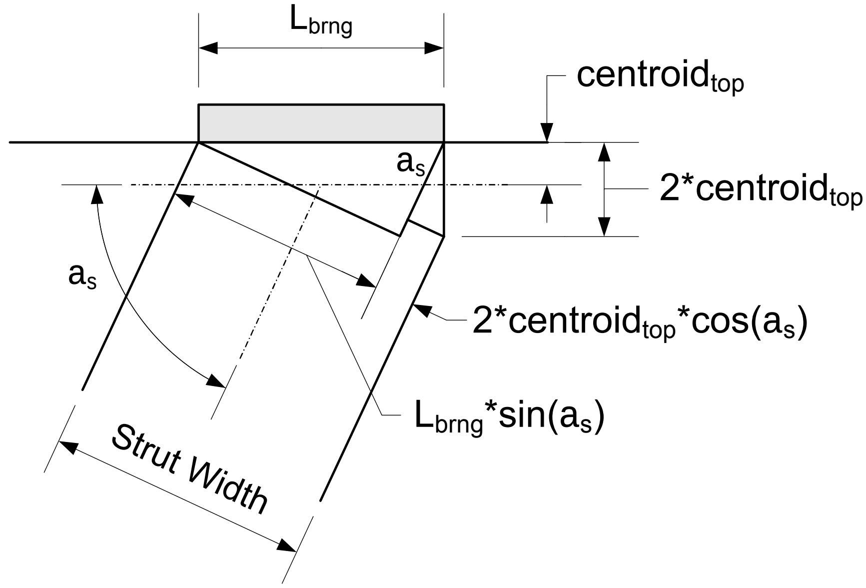


Figure E13-1.8-7
Compression Strut Width

Therefore at Node D, the thickness of the strut (t_{DI}) is:

$t_{DI} := L_{brng} \cdot \sin(\alpha_s) + 2 \cdot centroid_{top} \cdot \cos(\alpha_s)$ $\boxed{t_{DI} = 28.2}$ in

The effective compression strut width around each stirrup is:

$d_{bar11} := 1.410$ inches

$w_{ef} := 2 \cdot 6 \cdot d_{bar11}$ $\boxed{w_{ef} = 16.92}$ in

The effective spacing between the 4 legs of the stirrups is 13.5 inches, which is less than the value calculated above. Therefore, the entire cap width can be used for the effective strut width.

$w_{DI} := W_{cap} \cdot 12$ $\boxed{w_{DI} = 48}$ in

The nominal resistance (Pn_{DI}) of Strut DI is computed based on the limiting compressive stress, (f_{cu}), and the effective cross-section of the strut (Acn_{DI}) at the node face **LRFD [5.6.3.5]**.

$Acn_{DI} := t_{DI} \cdot w_{DI}$ $\boxed{Acn_{DI} = 1353.61}$ in^2

At Node D the node type is (CTT), and the surface where Strut DI meets the node is a strut to node interface. Analyze for crack control reinforcement being present.

At Node D, the capacity of Strut DI shall satisfy:

$Pu_{DI\_2} \le \phi c_{STM} \cdot (0.65 \cdot f'_c) \cdot Acn_{DI}$ --- (from Sect. 13.7.5)

$\phi c_{STM} := 0.7$ **LRFD [5.5.4.2]** $f'_c = 3.5$ ksi

The factored resistance is:

$Pr_{DI} := \phi c_{STM} \cdot (0.65 \cdot f'_c) \cdot Acn_{DI}$ $\boxed{Pr_{DI} = 2155.62}$ kips

$\boxed{|Pu_{DI\_2}| = 1471.41}$ kips

Is $Pr_{DI} \ge |Pu_{DI\_2}|$? $\boxed{check = \text{"OK"}}$

E13-1.8.7 Check the Anchorage of the Tension Ties

Tension ties shall be anchored in the nodal regions per **LRFD [5.6.3.4.2]**. The 9 No. 11 and 5 No. 10 longitudinal bars along the top of the pier cap must be developed at the inner edge of the bearing at Node E (the edge furthest from the end of the member). Based on (Figure E13-1.8-8), the embedment length that is available to develop the bar beyond the edge of the bearing is:

L_{devel} = (distance from cap end to Node E) + (bearing block width/2) - (cover)

$\boxed{L_{cap} = 46.5}$ feet (pier cap length)

$\boxed{S = 9.75}$ feet (girder spacing) $\boxed{ng = 5}$ (# girders)

$\boxed{L_{brng} = 26}$ inches (bearing block width)

$\boxed{Cover_{cp} = 2.5}$ inches (conc. cover)

$$L_{devel} := \frac{L_{cap} - S \cdot (ng - 1)}{2} \cdot 12 + \frac{L_{brng}}{2} - Cover_{cp} \qquad \boxed{L_{devel} = 55.5} \text{ in}$$

The basic development length for straight No. 11 and No. 10 bars with spacing less than 6", As(provided)/As(required) < 2, uncoated top bar, per (Wis Bridge Manual Table 9.9-1) is:

$\boxed{L_{d11} := 9.5}$ ft $\boxed{L_{d11} \cdot 12 = 114}$ in

$\boxed{L_{d10} := 7.75}$ ft $\boxed{L_{d10} \cdot 12 = 93}$ in

Therefore, there is not sufficient development length for straight bars. Check the hook development length. The base hook development length for 90° hooked No.11 and #10 bars per **LRFD [5.11.2.4]** is:

$$L_{hb11} := \frac{38.0 \cdot d_{bar11}}{\sqrt{f'_c}} \qquad \boxed{L_{hb11} = 28.64} \text{ in}$$

$$L_{hb10} := \frac{38.0 \cdot d_{bar10}}{\sqrt{f'_c}}$$ $\boxed{L_{hb10} = 25.8}$ in

The length available is greater than the base hook development length, therefore the reduction factors do not need to be considered. Hook both the top 9 bars and the bottom layer 5 bars. The remaining 4 bottom layer bars can be terminated 7.75 feet from the inside edge of the bearings at girders 2 and 4, which will allow all bars to be fully developed at this inside edge.

In addition, the tension ties must be spread out sufficiently in the effective anchorage area so that the compressive force on the back face of a CCT Node produced by the development of the ties through bond stress, does not exceed the factored resistance **LRFD [5.6.3.5]**.

Following the steps in E13-1.8.5, we can calculate the nominal resistance based on the limiting compressive stress, (f_{cu}), and the effective cross-section of the back face (Acn_E) at Node E. Analyze for crack control reinforcement being present.

The centroid of the tension ties is $\boxed{centroid_{top} = 5.5}$ inches below the top of the pier cap.

Therefore, the thickness (t_{DE}), and the width (w_{DE}) at the back face are:

$t_{DE} := 2 \cdot centroid_{top}$ $\boxed{t_{DE} = 11.0}$ in

$w_{DE} := W_{cap} \cdot 12$ $\boxed{w_{DE} = 48}$ in

$Acn_E := t_{DE} \cdot w_{DE}$ $\boxed{Acn_E = 528}$ in^2

$\boxed{Pu_{DE\_1} = 800.79}$ kips 1-lane loaded controls (Fig. E13-1.8-3)

The capacity at the back face of Node E shall satisfy:

$Pu_{DE\_1} \le \phi \cdot (0.70 \cdot f'_c) \cdot Acn_E$ --- (from Sect. 13.7.5)

$\phi = 0.9$ **LRFD [5.5.4.2]** $f'_c = 3.5$ ksi

The factored resistance is:

$Pr_{DE} := \phi \cdot (0.70 \cdot f'_c) \cdot Acn_E$ $\boxed{Pr_{DE} = 1164.24}$ kips

Is $Pr_{DE} \ge Pu_{DE\_1}$? $\boxed{check = \text{"OK"}}$

Because the compressive force on the backface is produced by development of reinforcement, the check as shown above is not necessary **LRFD [5.6.3.5.3b]**.

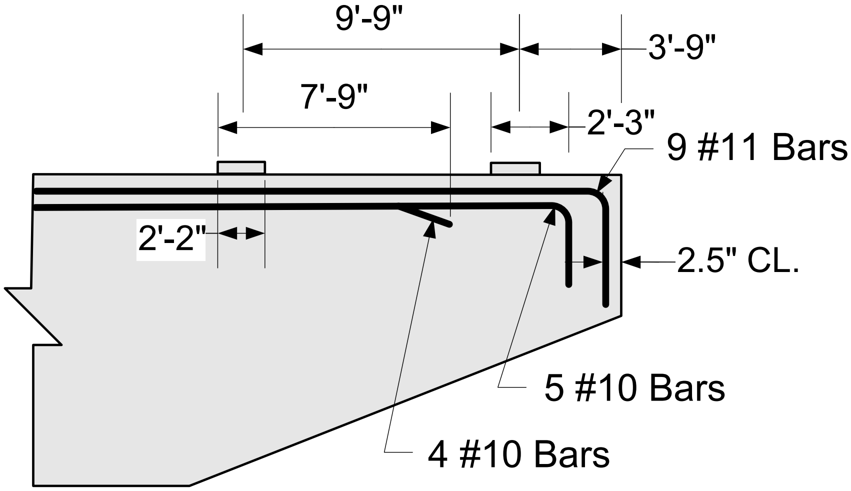


Figure E13-1.8-8
Anchorage of Tension Tie

E13-1.8.8 Provide Crack Control Reinforcement

In the pier cap, the minimum area of crack control reinforcement (As_{crack}) is equal to 0.003 times the width of the member (W_{cap}), and the spacing of the reinforcement (s_v, s_h) in each direction. The spacing of the bars in these grids must not exceed the smaller of d/4 or 12 inches, **LRFD [5.6.3.6]**.

$W_{cap} = 4.0$ ft

d/4 > 12" , therefore s_v and s_h = 12"

$As_{crack} := 0.003 \cdot (12) \cdot W_{cap} \cdot 12$ $\boxed{As_{crack} = 1.73}$ in^2

For horizontal reinforcement:

Use 4 - No. 7 horizontal bars at 12 inch spacing in the vertical direction - (Option 1)

$As_{No7} := 0.6013$ $\boxed{4 \cdot As_{No7} = 2.41}$ in^2

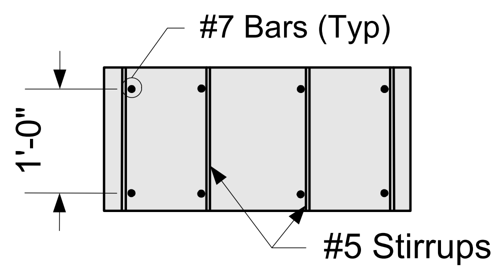


Figure E13-1.8-9
Crack Control Reinforcement - Option 1

OR: If we assume 6-inch vertical spacing - (Option 2)

$As_{crack} := 0.003 \cdot (6) \cdot W_{cap} \cdot 12$ $\boxed{As_{crack} = 0.86}$ in^2

Using 2 - No. 7 horiz. bars at 6 inch spacing $\boxed{2 \cdot As_{No7} = 1.2}$ in^2

Is $2 \cdot As_{No7} \geq As_{crack}$? $\boxed{check = \text{"OK"}}$

Therefore, No. 7 bars at 6" vertical spacing, placed horizontally on each side of the cap will satisfy this criteria.

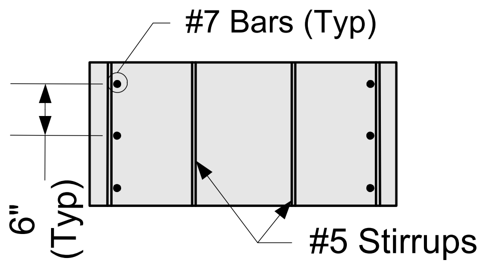


Figure E13-1.8-10
Crack Control Reinforcement - Option 2

This 6-inch spacing for the (No. 7 bars), is also used along the bottom of the cap for temperature and shrinkage reinforcement.

For vertical reinforcement:

The stirrups are spaced at, $\boxed{s_{stir} = 8}$ inches. Therefore the required crack control reinforcement within this spacing is:

$As_{crack2} := 0.003 \cdot (s_{stir}) \cdot W_{cap} \cdot 12$ $\boxed{As_{crack2} = 1.15}$ in^2

4 legs of No.5 stirrups at $\boxed{s_{stir} = 8}$ inch spacing in the horizontal direction

$\boxed{4 \cdot As_{No5} = 1.23}$ in^2

Is $4 \cdot As_{No5} \geq As_{crack2}$? $\boxed{check = \text{"OK"}}$

Therefore, pairs of (No. 5 bar) double-legged stirrups at 8" horizontal spacing will satisfy this criteria.

E13-1.8.9 Summary of Cap Reinforcement

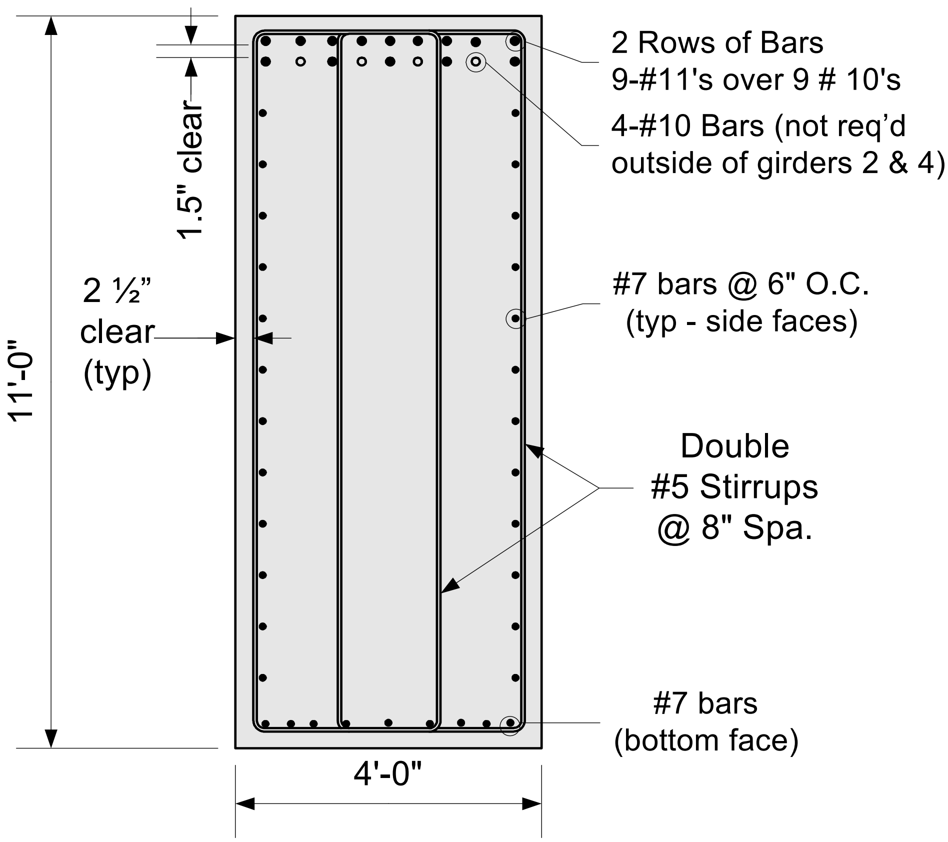


Figure E13-1.8-11
Pier Cap Design Summary

E13-1.9 Design Pier Column

As stated in E13-1.7, the critical section in the pier column is where the column meets the footing, or at the column base. The governing force effects and their corresponding limit states were determined to be:

Strength V

$\boxed{Ax_{colStrV} = 2099.51}$ kips

$\boxed{MuT_{colStrV} = 8315.32}$ kip-ft

$\boxed{MuL_{colStrV} = 2369.38}$ kip-ft

Strength III

$\boxed{VuT_{col} = 49.86}$ kips

Strength V

$\boxed{VuL_{col} = 109.25}$ kips

A preliminary estimate of the required section size and reinforcement is shown in Figure E13-1.9-1.

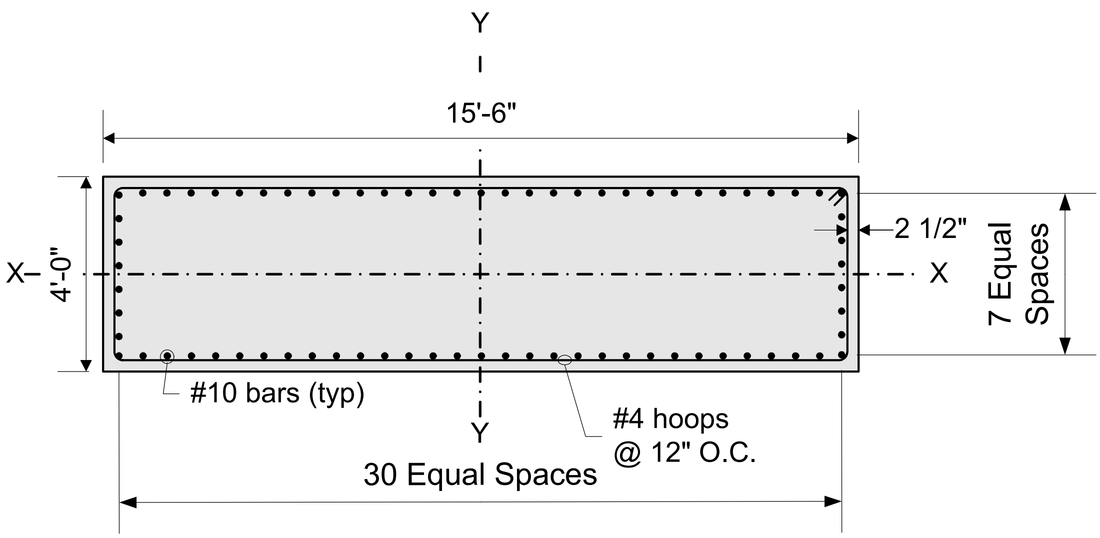


Figure E13-1.9-1
Preliminary Pier Column Design

E13-1.9.1 Design for Axial Load and Biaxial Bending (Strength V):

The preliminary column reinforcing is show in Figure E13-1.9-1 and corresponds to #10 bars equally spaced around the column perimeter. **LRFD [5.7.4.2]** prescribes limits (both maximum and minimum) on the amount of reinforcing steel in a column. These checks are performed on the preliminary column as follows:

$Num\_bars := 74$ $bar\_area10 := 1.27$ in^2 $bar\_dia10 := 1.27$ in

$A_{s\_col} := (Num\_bars) \cdot (bar\_area10)$ $\boxed{A_{s\_col} = 93.98}$ in^2

$A_{g\_col} := (W_{col}) \cdot (L_{col}) \cdot 12^2$ $\boxed{A_{g\_col} = 8928}$ in^2

$\frac{A_{s\_col}}{A_{g\_col}} = 0.0105$ $0.0105 \le 0.08$ (max. reinf. check) OK

$\frac{0.135 \cdot f'_c}{f_y} = 0.008$ (but need not be greater than 0.015) $0.0105 \geq 0.008$ (min. reinf. check) OK

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

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

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

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

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

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

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

The slenderness ratio for each axis now follows:

$$K_x := 2.1$$

$$K_y := 1.0$$

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

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

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

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

"X-X" axis).

In computing the amplification factor that is applied to the longitudinal moment, which is the end result of the slenderness effect, the column stiffness (EI) about the "X-X" axis must be defined. In doing so, the ratio of the maximum factored moment due to permanent load to the maximum factored moment due to total load must be identified (β_d).

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

$$\beta_d := 0$$

$$E_c := 33000 \cdot w_c^{1.5} \cdot \sqrt{f'_c}$$ **LRFD [C5.4.2.7]** $\boxed{E_c = 3587}$ ksi

$\boxed{E_s = 29000.00}$ ksi

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

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

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

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

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

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

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

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

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

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

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

$As_{col} := 2.53$ in^2 per foot, based on #10 bars at 6-inch spacing

$b := 12$ inches $\alpha_1 := 0.85$ (for $f'_c < 10.0$ ksi) **LRFD [5.7.2.2]**

$$a := \frac{As_{col} \cdot f_y}{\alpha_1 \cdot f'_c \cdot b} \qquad \boxed{a = 4.25} \quad \text{inches}$$

$\beta_1 := 0.85$

$$c := \frac{a}{\beta_1} \qquad \boxed{c = 5.00} \quad \text{inches}$$

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

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

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

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

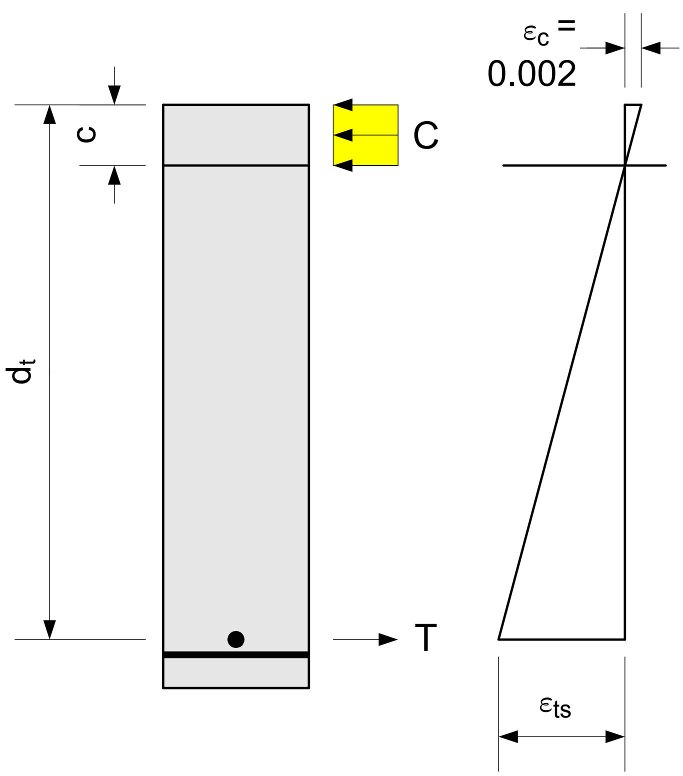


Figure E13-1.9-2
Strain Limit Tension Control Check

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

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

$\phi_t := 0.9$

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

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

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

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

$$P_{u\_col} := Ax_{colStrV} \qquad \boxed{P_{u\_col} = 2099.51} \quad \text{kips}$$

$$M_{ux} := MuL_{colStrV} \cdot \delta_s \qquad \boxed{M_{ux} = 2471.35} \quad \text{kip-ft}$$

$$M_{uy} := MuT_{colStrV} \qquad \boxed{M_{uy} = 8315.32} \quad \text{kip-ft}$$

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

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

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

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

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

$$\boxed{P_{u\_col} = 2099.51} \quad \text{kips} \qquad P_{u\_col} < 2343.6K$$

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

$\boxed{M_{ux} = 2471.35}$ kip-ft $\boxed{M_{uy} = 8315.32}$ kip-ft

The resultant moment equals:

$Mu := \sqrt{M_{ux}^2 + M_{uy}^2}$ $\boxed{Mu = 8674.8}$ kip-ft

$M_r := 24052.3$ kip-ft

$\boxed{\frac{Mu}{M_r} = 0.36}$ $0.36 \le 1.0$ OK

The factored flexural resistances shown above, M_r, was obtained by the use of commercial software. This value is the resultant flexural capacity assuming that no axial load is present. Consistent with this, the phi-factor for flexure (0.90) was used in obtaining the factored resistance from the factored nominal strength.

Although the column has a fairly large excess flexural capacity, a more optimal design will not be pursued per the discussion following the column shear check.

E13-1.9.2 Design for Shear (Strength III and Strength V)

The maximum factored transverse and longitudinal shear forces were derived in E13-1.7 and are as follows:

$\boxed{VuT_{col} = 49.86}$ kips (Strength III)

$\boxed{VuL_{col} = 109.25}$ kips (Strength V)

These maximum shear forces do not act concurrently. Although a factored longitudinal shear force is present in Strength III and a factored transverse shear force is present in Strength V, they both are small relative to their concurrent factored shear. Therefore, separate shear designs can be carried out for the longitudinal and transverse directions using only the maximum shear force in that direction.

For the pier column of this example, the maximum factored shear in either direction is less than one-half of the factored resistance of the concrete. Therefore, shear reinforcement is not required. This is demonstrated for the longitudinal direction as follows:

$b_v := L_{col} \cdot 12$ $\boxed{b_v = 186}$ in

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

Conservatively, d_v may be calculated as shown below, **LRFD [5.8.2.9]**.

$\boxed{d_v := (0.72) \cdot (h)}$ $\boxed{d_v = 34.56}$ in

The above calculation for d_v is simple to use for columns and generally results in a conservative estimate of the shear capacity.

$\beta := 2.0$ $\theta := 45deg$ $\lambda := 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]**

The nominal concrete shear strength is:

$V_c := 0.0316 \cdot \beta \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v$ **LRFD [5.8.3.3]** $\boxed{V_c = 760.04}$ kips

The nominal shear strength of the column is the lesser of the following two values:

$V_{n1} := V_c$ $\boxed{V_{n1} = 760.04}$ kips

$V_{n2} := 0.25 \cdot f'_c \cdot b_v \cdot d_v$ $\boxed{V_{n2} = 5624.64}$ kips

$V_n := \min(V_{n1}, V_{n2})$ $\boxed{V_n = 760.04}$ kips

The factored shear resistance is:

$\phi_v := 0.90$

$V_r := \phi_v \cdot V_n$ $\boxed{V_r = 684.04}$ kips

$\boxed{\frac{V_r}{2} = 342.02}$ kips

$\boxed{VuL_{col} = 109.25}$ kips $\frac{V_r}{2} > VuL_{col}$

$\boxed{check = "OK"}$

It has just been demonstrated that transverse steel is not required to resist the applied factored shear forces. However, transverse confinement steel in the form of hoops, ties or spirals is required for compression members. In general, the transverse steel requirements for shear and confinement must both be satisfied per the Specifications.

It is worth noting that although the preceding design checks for shear and flexure show the column to be over designed, a more optimal column size will not be pursued. The reason for this is twofold: First, in this design example, the requirements of the pier cap dictate the column dimensions (a reduction in the column width will increase the moment in the pier cap). Secondly, a short, squat column such as the column in this design example generally has a relatively large excess capacity even when only minimally reinforced.

E13-1.9.3 Transfer of Force at Base of Column

The provisions for the transfer of forces and moments from the column to the footing are new to the AASHTO LRFD Specifications. In general, standard engineering practice for bridge piers automatically satisfies most, if not all, of these requirements.

In this design example, and consistent with standard engineering practice, all steel reinforcing bars in the column extend into, and are developed, in the footing (see Figure E13-1.12-1). This automatically satisfies the following requirements for reinforcement across the interface of the column and footing: A minimum reinforcement area of 0.5 percent of the gross area of the supported member, a minimum of four bars, and any tensile force must be resisted by the reinforcement. Additionally, with all of the column reinforcement extended into the footing, along with the fact that the column and footing have the same compressive strength, a bearing check at the base of the column and the top of the footing is not applicable.

In addition to the above, the Specifications require that the transfer of lateral forces from the pier to the footing be in accordance with the shear-transfer provisions of **LRFD [5.8.4]**. With the standard detailing practices for bridge piers previously mentioned (i.e., all column reinforcement extended and developed in the footing), along with identical design compressive strengths for the column and footing, this requirement is generally satisfied. However, for the sake of completeness, this check will be carried out as follows:

$A_{cv} := A_{g\_col}$	Area of concrete engaged in shear transfer.	$\boxed{A_{cv} = 8928}$	in^2
$A_{vf} := A_{s\_col}$	Area of shear reinforcement crossing the shear plane.	$\boxed{A_{vf} = 93.98}$	in^2

For concrete placed against a clean concrete surface, not intentionally roughened, the following values are obtained from **LRFD [5.8.4.3]**.

$c_{cv} := 0.075$ Cohesion factor, ksi

$\mu := 0.60$ Friction factor

$K_1 := 0.2$

$K_2 := 0.8$

The nominal shear-friction capacity is the smallest of the following three equations (conservatively ignore permanent axial compression):

$V_{nsf1} := c_{cv} \cdot A_{cv} + \mu \cdot A_{vf} \cdot f_y$ $\boxed{V_{nsf1} = 4052.88}$ kips

$V_{nsf2} := K_1 \cdot f'_c \cdot A_{cv}$ $\boxed{V_{nsf2} = 6249.6}$ kips

$V_{nsf3} := K_2 \cdot A_{cv}$ $\boxed{V_{nsf3} = 7142.4}$ kips

Define the nominal shear-friction capacity as follows:

$V_{nsf} := \min\left(V_{nsf1}, V_{nsf2}, V_{nsf3}\right)$ $\boxed{V_{nsf} = 4052.88}$ kips

The maximum applied shear was previously identified from the Strength V limit state:

$\boxed{VuL_{col} = 109.25}$ kips

It then follows:

$\boxed{\phi_V = 0.9}$

$\boxed{\phi_V \cdot (V_{nsf}) = 3647.59}$ kips

$\phi_V \cdot (V_{nsf}) \geq VuL_{col}$

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

As can be seen, a large excess capacity exists for this check. This is partially due to the fact that the column itself is over designed in general (this was discussed previously). However, the horizontal forces generally encountered with common bridges are typically small relative to the shear-friction capacity of the column (assuming all reinforcing bars are extended into the footing). In addition, the presence of a shear-key, along with the permanent axial compression from the bridge dead load, further increase the shear-friction capacity at the column/footing interface beyond that shown above.

E13-1.10 Design Pier Piles

The foundation system for the pier is a reinforced concrete footing on steel H-piles. The HP12x53 pile layout used for this pier foundation is shown in Figure E13-1.10-1.

Based on the given pile layout, the controlling limit states for the pile design were given in E13-1.7.3.

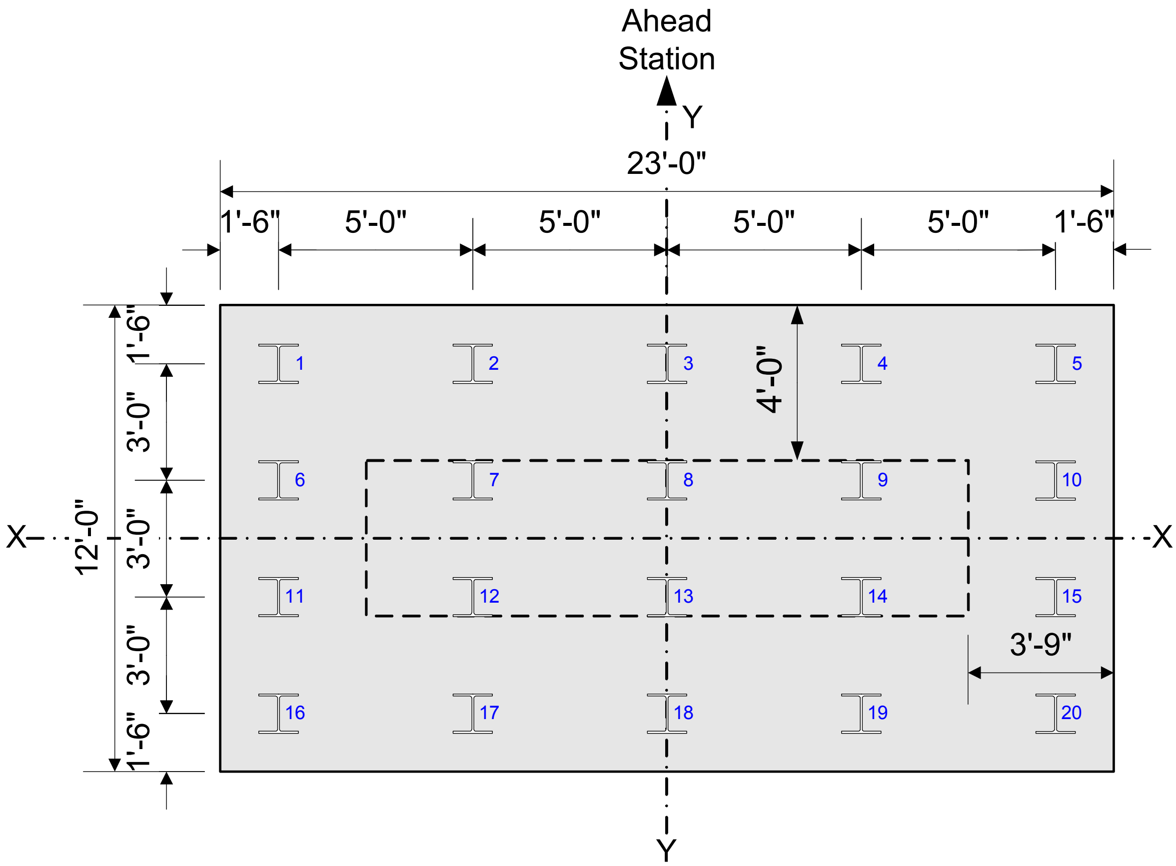


Figure E13-1.10-1
Pier Pile Layout

$N_p := 20$ Number of piles

$$S_{xx} := \frac{10 \cdot 4.5^2 + 10 \cdot 1.5^2}{4.5} \qquad \boxed{S_{xx} = 50} \quad ft^3$$

$$S_{yy} := \frac{8 \cdot 10^2 + 8 \cdot 5^2}{10} \qquad \boxed{S_{yy} = 100} \quad ft^3$$

Maximum pile reaction (Strength I):

$\boxed{\phi_t = 0.9}$

$\boxed{P_e = 56539.53}$ kips (from column design)

$\boxed{Pu2_{pile\_Str1} = 3179.17}$ kips

$\boxed{MuT2_{pile\_Str1} = 7836.85}$ kip-ft

$\boxed{MuL2_{pile\_Str1} = 1856.29}$ kip-ft

$$\delta_{pile\_StrI} := \frac{1}{1 - \left(\frac{Pu2_{pile\_Str1}}{\phi_t \cdot P_e}\right)}$$

$\boxed{\delta_{pile\_StrI} = 1.07}$

$$Pu_p := \frac{Pu2_{pile\_Str1}}{N_p} + \frac{MuT2_{pile\_Str1}}{S_{yy}} + \frac{MuL2_{pile\_Str1} \cdot \delta_{pile\_StrI}}{S_{xx}}$$

$\boxed{Pu_p = 276.93}$ kips

$$Pu_{p\_tons} := \frac{Pu_p}{2}$$

$\boxed{Pu_{p\_tons} = 138.46}$ tons

From Wis Bridge Manual, Section 11.3.1.17.6, the vertical pile resistance of HP12x53 pile is :

$\boxed{Pr_{12x53} = 110}$ tons $\boxed{check = \text{"No Good"}}$

$\boxed{Pr_{12x53\_PDA} = 143}$ tons $\boxed{check = \text{"OK"}}$

Note: PDA with CAPWAP is typically used when it is more economical than modified Gates. This example uses PDA with CAPWAP only to illustrate that vertical pile reactions are satisfied and to minimize example changes due to revised pile values. The original example problem was based on higher pile values than the current values shown in Chapter 11,Table 11.3-5.

Minimum pile reaction (Strength V):

$\boxed{Pu_{pile\_StrV} = 2179.55}$ kips

$\boxed{MuT_{pile\_StrV} = 7196.34}$ kip-ft

$\boxed{MuL_{pile\_StrV} = 2369.38}$ kip-ft

$$\delta_{pile\_StrV} := \frac{1}{1 - \left(\frac{Pu_{pile\_StrV}}{\phi_t \cdot P_e}\right)}$$

$\boxed{\delta_{pile\_StrV} = 1.04}$

$$Pu_{min\_p} := \frac{Pu_{pile\_StrV}}{N_p} - \frac{MuT_{pile\_StrV}}{S_{yy}} - \frac{MuL_{pile\_StrV} \cdot \delta_{pile\_StrV}}{S_{xx}}$$

$\boxed{Pu_{min\_p} = -12.49}$ kips

Capacity for pile uplift is site dependant. Consult with the geotechnical engineer for allowable values.

The horizontal pile resistance of HP12x53 pile from the soils report is :

$Hr_{12x53} := 14$ kips/pile

Pile dimensions in the transverse (xx) and longitudinal (yy) directions:

$B_{xx} := 12.05$ inches

$B_{yy} := 11.78$ inches

Pile spacing in the transverse and longitudinal directions:

$Spa_{xx} := 5.0$ feet $\boxed{\dfrac{Spa_{xx}}{\dfrac{B_{xx}}{12}} = 4.98}$ Say: 5B

$Spa_{yy} := 3.0$ feet $\boxed{\dfrac{Spa_{yy}}{\dfrac{B_{yy}}{12}} = 3.06}$ Say: 3B

Use the pile multipliers from **LRFD [Table 10.7.2.4-1]** to calculate the group resistance of the piles in each direction.

$Hr_{xx} := Hr_{12x53} \cdot 4 \cdot (1.0 + 0.85 + 0.70 \cdot 3)$

$\boxed{Hr_{xx} = 221.2}$ kips

$\boxed{HuT_{pileStrIII} = 49.86}$ kips

$Hr_{xx} \geq HuT_{pileStrIII}$

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

$Hr_{yy} := Hr_{12x53} \cdot 5 \cdot (0.7 + 0.5 + 0.35 \cdot 2)$

$\boxed{Hr_{yy} = 133}$ kips

$\boxed{HuL_{pileStrV} = 109.25}$ kips

$Hr_{yy} \geq HuL_{pileStrV}$

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

E13-1.11 - Design Pier Footing

In E13-1.7, the Strength I limit states was identified as the governing limit state for the design of the pier footing.

Listed below are the Strength I footing loads for one, two and three lanes loaded:

$\boxed{Pu1_{ftgStr1} = 2643.74}$ kips $\qquad$ $\boxed{Pu2_{ftgStr1} = 2928.7}$ kips

$\boxed{MuT1_{ftgStr1} = 7267.81}$ kip-ft $\qquad$ $\boxed{MuT2_{ftgStr1} = 7836.85}$ kip-ft

$\boxed{MuL1_{ftgStr1} = 1187.7}$ kip-ft $\qquad$ $\boxed{MuL2_{ftgStr1} = 1856.29}$ kip-ft

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

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

$\boxed{MuL3_{ftgStr1} = 2315.94}$ kip-ft

The longitudinal moment given above must be magnified to account for slenderness of the column (see E13-1.9). The computed magnification factor and final factored forces are:

$$\delta_{s1\_ftgStr1} := \frac{1}{1 - \left(\frac{Pu1_{ftgStr1}}{\phi_t \cdot P_e}\right)} \qquad \boxed{\delta_{s1\_ftgStr1} = 1.05}$$

$$\delta_{s2\_ftgStr1} := \frac{1}{1 - \left(\frac{Pu2_{ftgStr1}}{\phi_t \cdot P_e}\right)} \qquad \boxed{\delta_{s2\_ftgStr1} = 1.06}$$

$$\delta_{s3\_ftgStr1} := \frac{1}{1 - \left(\frac{Pu3_{ftgStr1}}{\phi_t \cdot P_e}\right)} \qquad \boxed{\delta_{s3\_ftgStr1} = 1.07}$$

$MuL1_{ftgStr1\delta} := \delta_{s1\_ftgStr1} \cdot MuL1_{ftgStr1}$ $\qquad$ $\boxed{MuL1_{ftgStr1\delta} = 1252.79}$ kip-ft

$MuL2_{ftgStr1\delta} := \delta_{s2\_ftgStr1} \cdot MuL2_{ftgStr1}$ $\qquad$ $\boxed{MuL2_{ftgStr1\delta} = 1969.65}$ kip-ft

$MuL3_{ftgStr1\delta} := \delta_{s3\_ftgStr1} \cdot MuL3_{ftgStr1}$ $\qquad$ $\boxed{MuL3_{ftgStr1\delta} = 2467.46}$ kip-ft

The calculations for the Strength I pile loads on the footing are calculated below for one, two and three lanes loaded.

$\boxed{N_p = 20}$ Number of piles

$\boxed{S_{xx} = 50}$ ft^3

$\boxed{S_{yy} = 100}$ ft^3

The following illustrates the corner pile loads for 2 lanes loaded:

$$Pu2_1 := \frac{Pu2_{ftgStr1}}{N_p} + \frac{MuT2_{ftgStr1}}{S_{yy}} + \frac{MuL2_{ftgStr1\delta}}{S_{xx}} \qquad \boxed{Pu2_1 = 264.2} \text{ kips}$$

$$Pu2_5 := \frac{Pu2_{ftgStr1}}{N_p} - \frac{MuT2_{ftgStr1}}{S_{yy}} + \frac{MuL2_{ftgStr1\delta}}{S_{xx}} \qquad \boxed{Pu2_5 = 107.46} \text{ kips}$$

$$Pu2_{16} := \frac{Pu2_{ftgStr1}}{N_p} + \frac{MuT2_{ftgStr1}}{S_{yy}} - \frac{MuL2_{ftgStr1\delta}}{S_{xx}} \qquad \boxed{Pu2_{16} = 185.41} \text{ kips}$$

$$Pu2_{20} := \frac{Pu2_{ftgStr1}}{N_p} - \frac{MuT2_{ftgStr1}}{S_{yy}} - \frac{MuL2_{ftgStr1\delta}}{S_{xx}} \qquad \boxed{Pu2_{20} = 28.67} \text{ kips}$$

Pile loads between the corners can be interpolated. Similar calculations for the piles for the cases of one, two and three lanes loaded produce the following results:

$$Pu1 = \begin{pmatrix} 229.92 & 193.58 & 157.24 & 120.9 & 84.56 \\ 213.22 & 176.88 & 140.54 & 104.2 & 67.86 \\ 196.51 & 160.17 & 123.84 & 87.5 & 51.16 \\ 179.81 & 143.47 & 107.13 & 70.79 & 34.45 \end{pmatrix}$$

$$Pu2 = \begin{pmatrix} 264.2 & 225.01 & 185.83 & 146.64 & 107.46 \\ 237.93 & 198.75 & 159.57 & 120.38 & 81.2 \\ 211.67 & 172.49 & 133.3 & 94.12 & 54.94 \\ 185.41 & 146.23 & 107.04 & 67.86 & 28.67 \end{pmatrix}$$

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

$\boxed{Pu1_{pile} = 229.92}$ kips $\boxed{Pu2_{pile} = 264.2}$ kips $\boxed{Pu3_{pile} = 251}$ kips

A conservative simplification is to use the maximum pile reaction for all piles when calculating the total moment and one way shear forces on the footing.

$Pu := \max(Pu1_{pile}, Pu2_{pile}, Pu3_{pile})$ $\boxed{Pu = 264.2}$ kips

E13-1.11.1 Design for Moment

The footing is designed for moment using the pile forces computed above on a per-foot basis acting on each footing face. The design section for moment is at the face of the column. The following calculations are based on the outer row of piles in each direction, respectively.

$L_{ftg\_xx} := L_{ftg}$ $\boxed{L_{ftg\_xx} = 23}$ feet

$L_{ftg\_yy} := W_{ftg}$ $\boxed{L_{ftg\_yy} = 12}$ feet

Applied factored load per foot in the "X" direction:

$Pu_{Mom\_xx} := Pu \cdot 5$ $\boxed{Pu_{Mom\_xx} = 1320.98}$ kips

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

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

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

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

$arm_{xx} := 2.5$ feet

$arm_{yy} := 2.25$ feet

The moment on a per foot basis is then:

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

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

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

Assume #8 bars:

$bar\_diam8 := 1.0$ inches

$bar\_area8 := 0.79$ in^2

$\boxed{f_y = 60}$ ksi

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

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

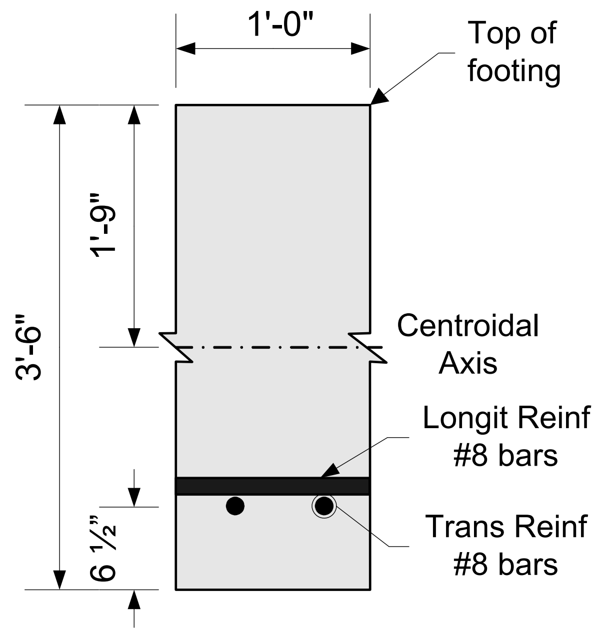


Figure E13-1.11-1
Footing Cracking Moment Dimensions

$f_r = 0.24 \cdot \lambda \sqrt{f'_c}$ = 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]** $\boxed{f_r = 0.45}$ ksi

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

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

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

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

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

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

1.33 times the factored controlling footing moment is:

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

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

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

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

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

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

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

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

$\phi_f := 0.90$

$\boxed{b = 12}$ in

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

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

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

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

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

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

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

Is $A_{sftg} \geq As_{ftg}$? $\boxed{check = \text{"OK"}}$

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

E13-1.11.2 Punching Shear Check

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

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

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

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

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

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

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

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

$\boxed{b = 12}$ in

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

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

Effective depth for each axis:

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

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

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

Effective shear depth for each axis:

$$T_{ftg} := A_{s\_ftg} \cdot f_y \qquad \boxed{T_{ftg} = 94.8} \quad \text{kips}$$

$$a_{ftg} := \frac{T_{ftg}}{\alpha_1 \cdot f'_c \cdot b} \qquad \boxed{a_{ftg} = 2.66} \quad \text{in}$$

$$d_{vx} := \max\left(d_{ex} - \frac{a_{ftg}}{2}, 0.9 \cdot d_{ex}, 0.72 \cdot h_{ftg}\right) \qquad \boxed{d_{vx} = 33.17} \quad \text{in}$$

$$d_{vy} := \max\left(d_{ey} - \frac{a_{ftg}}{2}, 0.9 \cdot d_{ey}, 0.72 \cdot h_{ftg}\right) \qquad \boxed{d_{vy} = 34.17} \quad \text{in}$$

Average effective shear depth:

$$d_{v\_avg} := \frac{d_{vx} + d_{vy}}{2} \qquad \boxed{d_{v\_avg} = 33.67} \quad \text{in}$$

With the average effective shear depth determined, the critical perimeter can be calculated as follows:

$$b_{col} := L_{col} \cdot 12 \qquad \boxed{b_{col} = 186} \quad \text{in}$$

$$t_{col} := W_{col} \cdot 12 \qquad \boxed{t_{col} = 48} \quad \text{in}$$

$$b_o := 2\left[b_{col} + 2 \cdot \left(\frac{d_{v\_avg}}{2}\right)\right] + 2 \cdot \left[t_{col} + 2 \cdot \left(\frac{d_{v\_avg}}{2}\right)\right] \qquad \boxed{b_o = 602.69} \quad \text{in}$$

The factored shear resistance to punching shear is the smaller of the following two computed values: **LRFD [5.13.3.6.3]**

$$\beta_c := \frac{b_{col}}{t_{col}} \qquad \boxed{\beta_c = 3.88}$$

λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$$V_{n\_punch1} := \left(0.063 + \frac{0.126}{\beta_c}\right) \cdot \lambda \sqrt{f'_c} \cdot (b_o) \cdot (d_{v\_avg}) \qquad \boxed{V_{n\_punch1} = 3626.41} \quad \text{kips}$$

$$V_{n\_punch2} := 0.126 \cdot \left(\lambda \sqrt{f'_c}\right) \cdot (b_o) \cdot (d_{v\_avg}) \qquad \boxed{V_{n\_punch2} = 4783.77} \quad \text{kips}$$

$$V_{n\_punch} := \min(V_{n\_punch1}, V_{n\_punch2}) \qquad \boxed{V_{n\_punch} = 3626.41} \quad \text{kips}$$

$$\phi_v = 0.9$$

$$V_{r\_punch} := \phi_v \cdot (V_{n\_punch}) \qquad \boxed{V_{r\_punch} = 3263.77} \quad \text{kips}$$

With the factored shear resistance determined, the applied factored punching shear load will be computed. This value is obtained by summing the loads in the piles that are outside of the critical perimeter. As can be seen in Figure E13-1.11-2, this includes Piles 1 through 5, 6, 10,11, 15, and 16 through 20. These piles are entirely outside of the critical perimeter. If part of a pile is inside the critical perimeter, then only the portion of the pile load outside the critical perimeter is used for the punching shear check, **LRFD [5.13.3.6.1]**.

$$\left(\frac{t_{col}}{2} + \frac{d_{v\_avg}}{2}\right) \cdot \frac{1}{12} = 3.4 \qquad \text{feet}$$

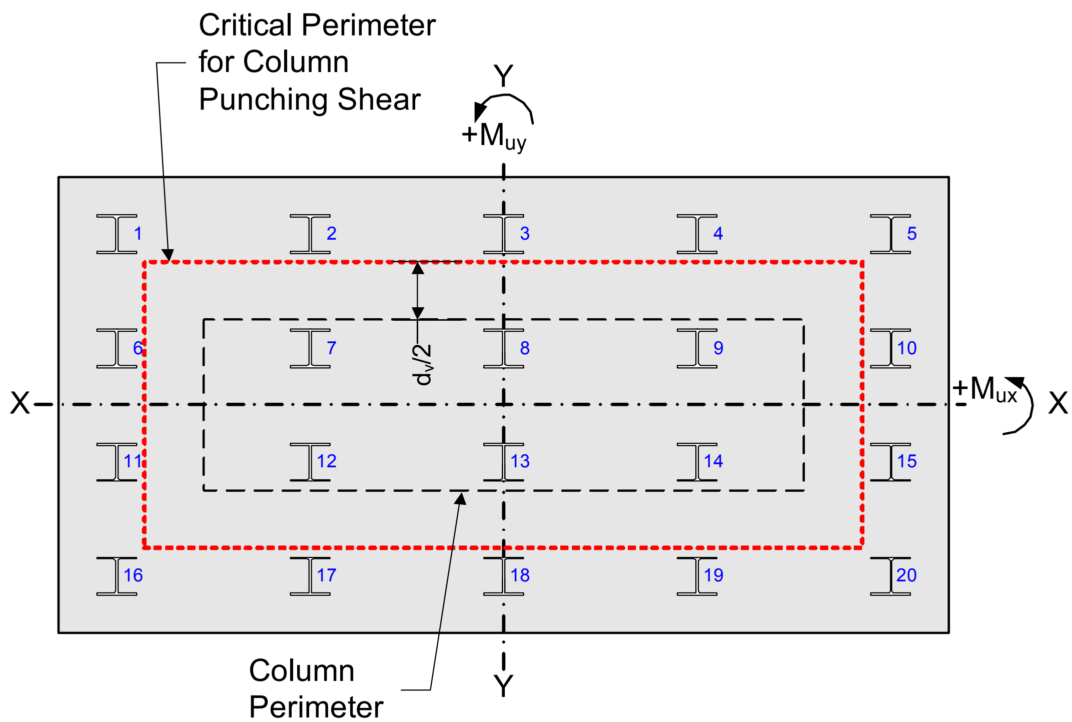


Figure E13-1.11-2
Critical Perimeter for Column Punching Shear

The total applied factored shear used for the punching shear check is the sum of the piles outside of the shear perimeter (1 through 5, 6, 10, 11, 15 and 16 through 20):

$$V_{u\_punch} := \max\left(Pu1_{punch\_col}, Pu2_{punch\_col}, Pu3_{punch\_col}\right)$$

$\boxed{V_{u\_punch} = 2187.26}$ kips

$\boxed{V_{r\_punch} = 3263.77}$ kips

$V_{u\_punch} \le V_{r\_punch}$

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

For <u>two-way action</u> around the <u>maximum loaded pile</u>, the pile critical perimeter, b_o, is located a minimum of $0.5d_v$ from the perimeter of the pile. If portions of the critical perimeter are located off the footing, that portion of the critical perimeter is limited by the footing edge.

Two-way action should be checked for the maximum loaded pile, The effective shear depth, d_v is the same as that used for the punching shear check for the column.

$V_{u2way} := Pu2_{pile}$ $\boxed{V_{u2way} = 264.2}$ kips

$\boxed{d_{v\_avg} = 33.67}$ in

$\boxed{0.5 \cdot d_{v\_avg} = 16.84}$ in

Two-way action or punching shear resistance for sections without transverse reinforcement can then be calculated as follows: **LRFD [5.13.3.6.3]**

λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$$V_n = \left(0.063 + \frac{0.126}{\beta_c}\right) \cdot \lambda \sqrt{f'_c} \cdot b_o \cdot d_v \le 0.126 \cdot \lambda \sqrt{f'_c} \cdot b_o \cdot d_v$$

$\boxed{B_{xx} = 12.05}$ in

$\boxed{B_{yy} = 11.78}$ in

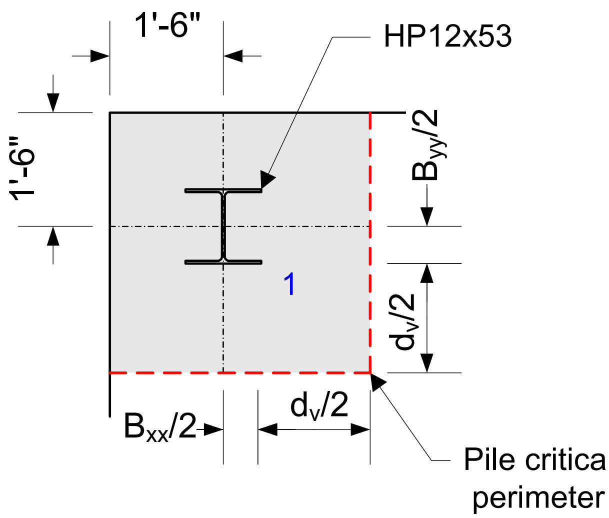


Figure E13-1.11-3
Pile Two-way Action Critical Perimeter

Since the critical section is outside of the footing, only include the portion of the shear perimeter that is located within the footing:

$b_{o\_xx} := 1.5 \cdot 12 + \frac{B_{xx}}{2} + \frac{d_{v\_avg}}{2}$ $\boxed{b_{o\_xx} = 40.86}$ in

$b_{o\_yy} := 1.5 \cdot 12 + \frac{B_{yy}}{2} + \frac{d_{v\_avg}}{2}$ $\boxed{b_{o\_yy} = 40.73}$ in

Ratio of long to short side of critical perimeter:

$\beta_{c\_pile} := \dfrac{b_{o\_xx}}{b_{o\_yy}}$ $\boxed{\beta_{c\_pile} = 1.003}$

$b_{o\_pile} := b_{o\_xx} + b_{o\_yy}$ $\boxed{b_{o\_pile} = 81.59}$ in

$V_{n\_pile1} := \left(0.063 + \dfrac{0.126}{\beta_{c\_pile}}\right) \cdot \lambda \sqrt{f'_c} \cdot (b_{o\_pile}) \cdot (d_{v\_avg})$ $\boxed{V_{n\_pile1} = 969.24}$ kips

$V_{n\_pile2} := 0.126 \cdot \left(\lambda \sqrt{f'_c}\right) \cdot (b_{o\_pile}) \cdot (d_{v\_avg})$ $\boxed{V_{n\_pile2} = 647.59}$ kips

$V_{n\_pile} := \min(V_{n\_pile1}, V_{n\_pile2})$ $\boxed{V_{n\_pile} = 647.59}$ kips

$\phi_v = 0.9$

$V_{r\_pile} := \phi_v \cdot (V_{n\_pile})$ $\boxed{V_{r\_pile} = 582.83}$ kips

$\boxed{V_{u2way} = 264.2}$ kips

$V_{r\_pile} \geq V_{u2way}$

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

E13-1.11.3 One Way Shear Check

Design for <u>one way shear</u> in both the <u>transverse</u> and <u>longitudinal directions</u>.

For one way action in the pier footing, in accordance with **LRFD[5.13.3.6.1]** & **[5.8.3.2]** the critical section is taken as the larger of:

$0.5 \cdot d_v \cdot \cot\theta$ or d_v

$\theta := 45\text{deg}$

The term d_v is calculated the same as it is for the punching shear above:

$\boxed{d_{vx} = 33.17}$ in

$\boxed{d_{vy} = 34.17}$ in

Now the critical section can be calculated:

$dv_{xx} := \max(0.5 \cdot d_{vx} \cdot \cot(\theta), d_{vx})$ $\boxed{dv_{xx} = 33.17}$ in

$dv_{yy} := \max(0.5 \cdot d_{vy} \cdot \cot(\theta), d_{vy})$ $\boxed{dv_{yy} = 34.17}$ in

Distance from face of column to CL of pile in longitudinal and transverse directions:

$\boxed{arm_{xx} = 2.5}$ feet

$\boxed{arm_{yy} = 2.25}$ feet

Distance from face of column to outside edge of pile in longitudinal and transverse directions:

$\boxed{arm_{xx} \cdot 12 + \frac{B_{yy}}{2} = 35.89}$ in > d_{vx}, design check required

$\boxed{arm_{yy} \cdot 12 + \frac{B_{xx}}{2} = 33.02}$ in < d_{vy}, no design check required

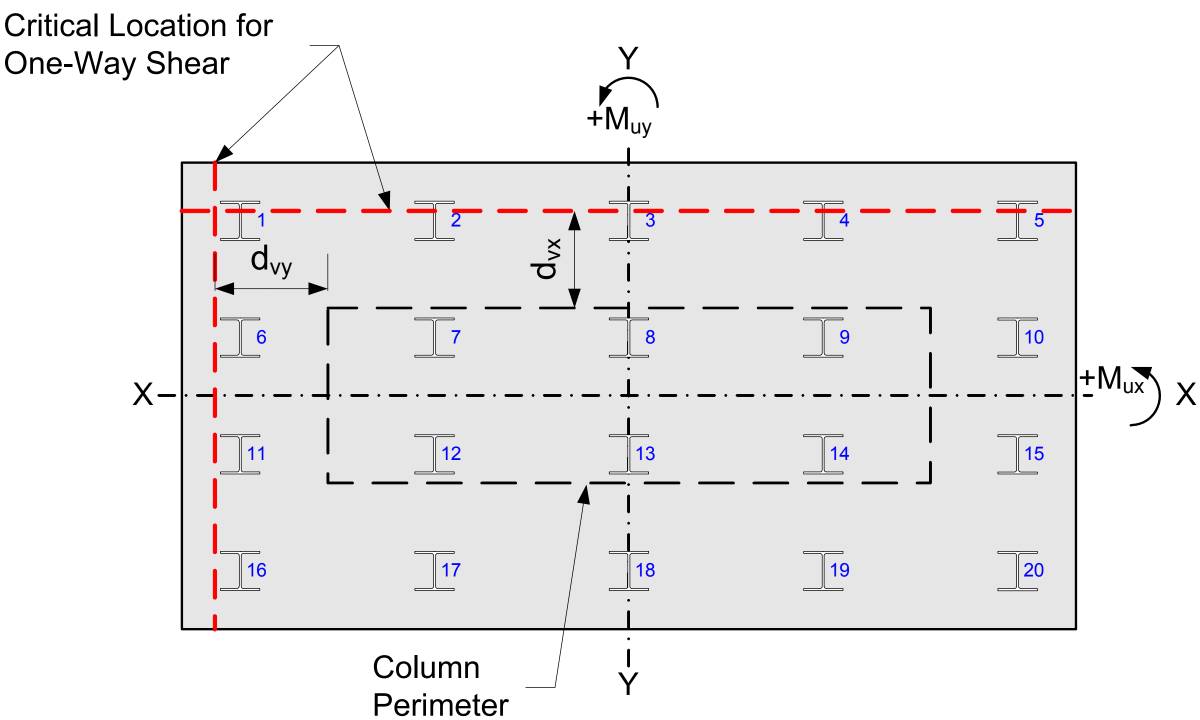


Figure E13-1.11-4
Critical Section for One-Way Shear

Portion of pile outside of the critical section for one way shear in the longitudinal direction:

$$b_{xx} := arm_{xx} \cdot 12 + \frac{B_{yy}}{2} - d_{vx}$$

$\boxed{b_{xx} = 2.72}$ inches

The load applied to the critical section will be based on the proportion of the pile located outside of the critical section. As a conservative estimate, the maximum pile reaction will be assumed for all piles.

$\boxed{Pu = 264.2}$ kips

$Pu_{1wayx} := Pu \cdot 5$ $\boxed{Pu_{1wayx} = 1320.98}$ kips

$Vu_{1wayx} := Pu_{1wayx} \cdot \frac{b_{xx}}{B_{yy}}$ $\boxed{Vu_{1wayx} = 304.76}$ kips

The nominal shear resistance shall be calculated in accordance with **LRFD [5.8.3.3]** and is the lesser of the following:

$\beta_{1way} := 2.0$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$b_v := L_{ftg} \cdot 12$ $\boxed{b_v = 276}$ inches

$V_{n\_1way1} := 0.0316 \cdot \beta_{1way} \cdot \lambda \sqrt{f'_c} \cdot (b_v) \cdot (d_{vx})$ $\boxed{V_{n\_1way1} = 1082.52}$ kips

$V_{n\_1ay2} := 0.25 \cdot (f'_c) \cdot (b_v) \cdot (d_{vx})$ $\boxed{V_{n\_1ay2} = 8011.1}$ kips

$V_{n\_1way} := \min(V_{n\_1way1}, V_{n\_1ay2})$ $\boxed{V_{n\_1way} = 1082.52}$ kips

$\phi_v = 0.9$

$V_{r\_1way} := \phi_v \cdot (V_{n\_1way})$ $\boxed{V_{r\_1way} = 974.27}$ kips

$\boxed{Vu_{1wayx} = 304.76}$ kips

$V_{r\_1way} \geq Vu_{1wayx}$

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

E13-1.12 Final Pier Schematic

Figure E13-1.12-1 shows the final pier dimensions along with the required reinforcement in the pier cap and column.

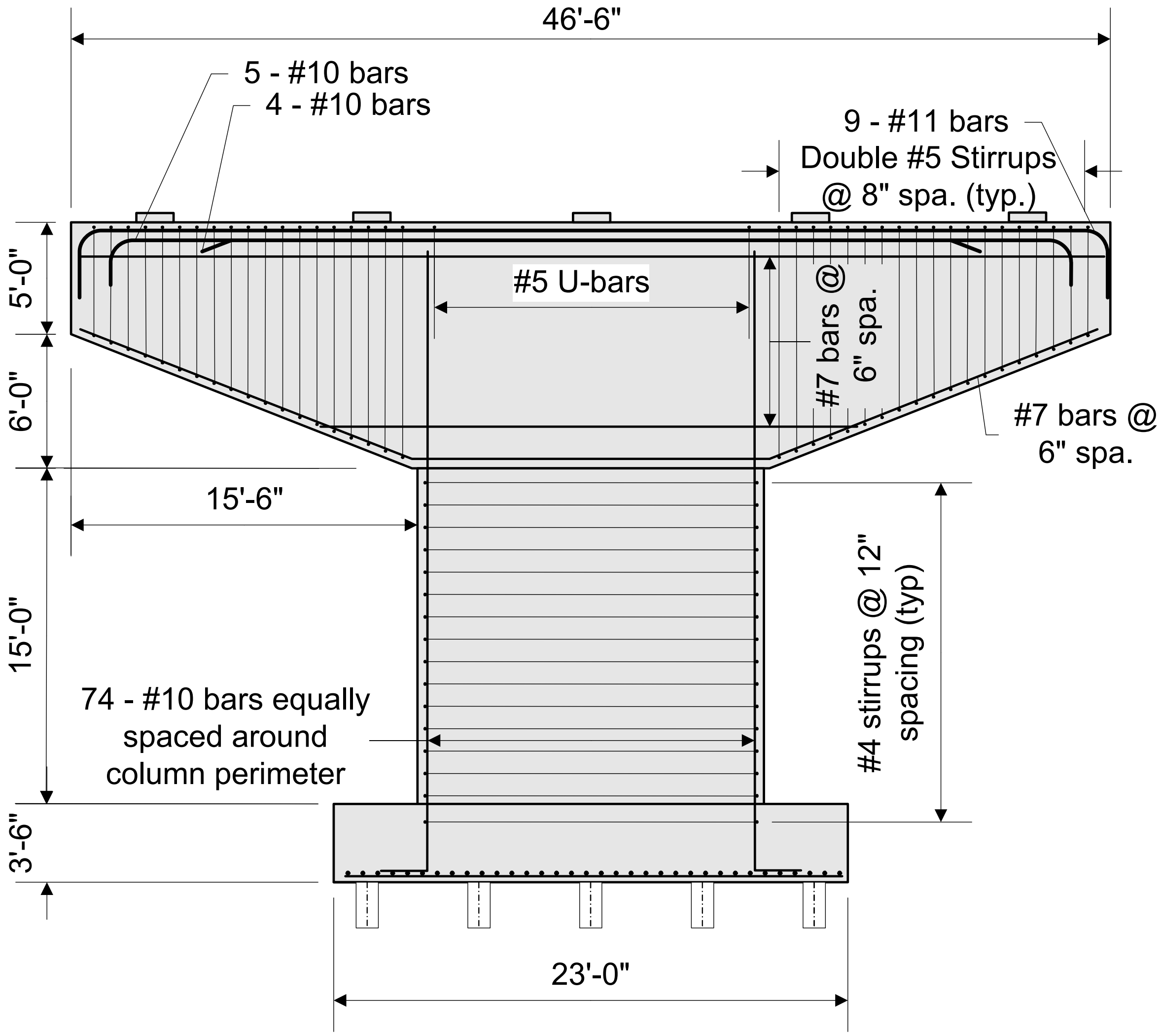


Figure E13-1.12-1
Final Pier Design

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Table of Contents

14.1 Introduction

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

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

WisDOT policy item:

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

14.1.1 Wall Development Process

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

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

Based on the SSR, a Geotechnical site investigation (see Chapter 10 – Geotechnical Investigation) may be required to determine foundation and retained soil properties. A hydraulic analysis is also conducted, if required, to asses scour potential. The Geotechnical investigation generally includes a subsurface and laboratory investigation. For the departmental-designed walls, the Bureau of Technical Services, Geotechnical Engineering Unit can recommend the scope of soil exploration needed and provide/recommend bearing resistance, overall stability, and settlement of walls based on the geotechnical exploration results. These Geotechnical recommendations are presented in a Site Investigation Report.

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

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

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

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

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

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

14.1.1.1 Wall Numbering System

Permanent retaining walls should be identified by a Wall Number, R-XX-XXX, as assigned by the Region using the conventions described below. For a continuous wall consisting of various wall types, such as a secant pile wall followed by a solider pile wall, a unique Wall Number should be assigned to each wall type segment. These unique Wall Numbers will be beneficial for inspection purposes. Unit numbers, such as R-XX-XXX-1, may be assigned in lieu of unique Wall Numbers – contact the Regional Ancillary Program Manager for approval. Additional coordination with the Region is necessary for assigning additional Wall Numbers and/or Unit Numbers. Discontinuities at wall facings (e.g. stairwells, staged construction, tiers, or changes to external loads) do not require unique Wall Numbers if the leveling pad or footing is continuous between the completed wall segments. For solider pile walls with anchored and non-anchored segments, unique Wall Numbers are not required for each segment.

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

- Proprietary retaining walls (e.g., modular block MSE walls)

  - MSE walls having a maximum height of less than 5.5 ft. measured from the bottom of wall or top of leveling pad to top of wall are deemed to be “minor retaining walls” and do not require an R number. Refer to *FDM 11-55-5.2* for more information.

  - Modular block gravity walls having a maximum height of less than 4.0 ft. measured from the bottom of wall or top of leveling pad to top of wall are deemed to be “minor retaining walls” and do not require an R number. Refer to *FDM 11-55-5.2* for more information.

- <u>Non-proprietary walls</u> (e.g., sheet pile walls):

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

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

14.2 Wall Types

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

Conventional Walls

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

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

Permanent or Temporary Walls

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

Fill Walls or Cut Walls

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

Bottom-up or Top-down Constructed Walls

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

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.

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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.
- All preliminary designs starting after October 1, 2017.

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 were based on significant changes in the vehicle fleet, the emergence of many new barrier designs, increased interest in matching safety performance to levels of roadway utilization, new policies requiring the use of safety belts, and advances in computer simulation and other evaluation methods.

NCHRP Report 350 differs from NCHRP Report 230 in the following ways: it is presented in all-metric documentation, it provides a wider range of test procedures to permit safety performance evaluations for a wider range of barriers, it uses a pickup truck as the standard test vehicle in place of a passenger car, it defines other supplemental test vehicles, it includes a broader range of tests to provide a uniform basis for establishing warrants for the application of roadside safety hardware that consider the levels of use of the roadway facility, it includes guidelines for selection of the critical impact point for crash tests on redirecting-type safety hardware, it provides information related to enhanced measurement techniques related to occupant risk, and it reflects a critical review of methods and technologies for safety-performance evaluation.

In May of 1997, a memorandum from Dwight A. Horne, the FHWA Chief of the Federal-Aid and Design Division, on the subject of “Crash Testing of Bridge Railings” was published. This memorandum identified 68 crash-tested bridge rails, consolidated earlier listings, and established tentative equivalency ratings that related previous NCHRP Report 230 testing to NCHRP Report 350 test levels.

In 2009, AASHTO published the *Manual for Assessing Safety Hardware* (MASH). MASH is an update to, and supersedes, NCHRP Report 350 for the purposes of evaluating new safety hardware devices. AASHTO and FHWA jointly adopted an implementation plan for MASH that stated that all highway safety hardware accepted prior to the adoption of MASH – using criteria contained in NCHRP Report 350 – may remain in place and may continue to be manufactured and installed. In addition, highway safety hardware accepted using NCHRP Report 350 criteria is not required to be retested using MASH criteria. However, new highway safety hardware not previously evaluated must utilize MASH for testing and evaluation. MASH represents an update to crash testing requirements based primarily on changes in the vehicle fleet.

All bridge railings as detailed in the Wisconsin LRFD Bridge Standard Detail Drawings in Chapter 30 are approved for use on WisDOT projects. In order to use railings other than Bureau of Structures Standards, the railings must conform to MASH or must be crash tested rails which are available from the FHWA office. Any railing not in the Standards must be approved by the Bureau of Structures. Any railings that are not crash tested must be reviewed by FHWA when they are used on a bridge, culvert, retaining wall, etc.

WisDOT and FHWA policy states that railings that meet the criteria for Test Level 3 (TL-3) or greater shall be used on NHS roadways and all functional classes of Wisconsin structures (Interstate Highways, United States Highways, State Trunk Highways, County Trunk Highways, and Local Roadways) where the design speed exceeds 45 mph. Railings that meet Test Level 2 (TL-2) criteria may be used on non-NHS roadways where the design speed is 45 mph or less.

There may be unique situations that may require the use of a MASH crash-tested railing of a different Test Level; a railing design using an older crash test methodology; or a modified railing system based on computer modeling, component testing, and or expert opinion. These unique situations will require an exception to be granted by the Bureau of Project Development and/or the Bureau of Structures. It is recommended that coordination of these unique situations occur early in the design process.

30.2 Railing Application

The primary purpose of bridge railings shall be to contain and redirect vehicles and/or pedestrians using the structure. In general, there are three types of bridge railings – Traffic Railings, Combination Railings, and Pedestrian Railings. The following guidelines indicate the typical application of each railing type:

1. Traffic Railings shall be used when a bridge is used exclusively for highway traffic.

   Traffic Railings can be composed of, but are not limited to: single slope concrete parapets, sloped face concrete parapets, vertical face concrete parapets, tubular steel railings, and timber railings.

2. Combination Railings can be used concurrently with a raised sidewalk on roadways with a design speed of 45 mph or less.

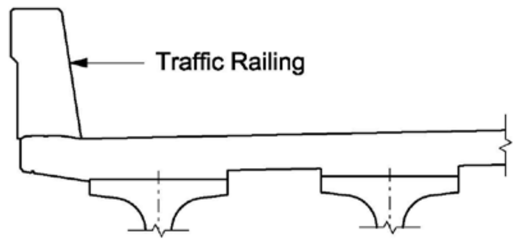
   Combination Railings can be composed of, but are not limited to: single slope concrete parapets with chain link fence, vertical face concrete parapets with tubular steel railings such as type 3T, and aesthetic concrete parapets with combination type C1-C6 railings.

3. Pedestrian Railings can be used at the outside edge of a bridge sidewalk when a Traffic Railing is used concurrently to separate highway and pedestrian traffic.

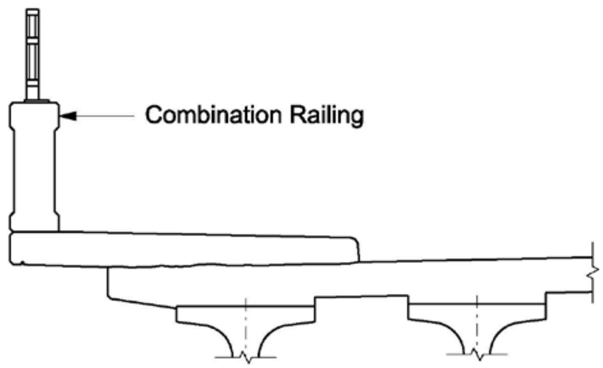
   Pedestrian Railings can be composed of, but are not limited to: chain link fence, tubular screening, vertical face concrete parapets with combination type C1-C6 or type 3T railings, and single slope concrete parapets.

See Figure 30.2-1 below for schematics of the three typical railing types.

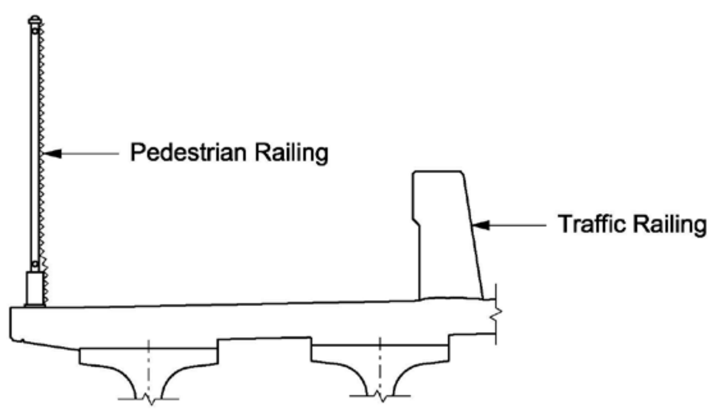
Note that the railing types shown in Figure 30.2-1 shall be employed as minimums. At locations where a Traffic Railing is used at the traffic side of a sidewalk at grade, a Combination Railing may be used at the edge of deck in lieu of a Pedestrian Railing. At locations where a Combination Railing is used at the exterior edge of a raised sidewalk, a Traffic Railing may be used as an alternative as long as the requirements for Pedestrian Railings are met.



Traffic Railing
All Design Speeds



Combination Railling
Design Speeds of 45 mph or Less



Pedestrian Railing
All Design Speeds

Figure 30.2-1
Bridge Railing Types

The application of bridge railings shall comply with the following guidance:

1. All bridge railings shall conform to **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. The vertical face parapet "A" can be used for all design speeds. The vertical face parapet is recommended for use as a Combination Railing on raised sidewalks or as a Traffic Railing where the design speed is 45 mph or less. If the structure has a raised sidewalk on one side only, a sloped parapet should be used on the side opposite of the sidewalk. For design speeds exceeding 45 mph, at locations where the parapet is protected by a Traffic Railing between the roadway and a sidewalk at grade, the vertical face parapet can be used as a Pedestrian Railing. Under some circumstances, the vertical face parapet "A" can be used as a Traffic Railing for design speeds exceeding 45 mph with the approval of the Bureau of Structures Development Section. The vertical face parapet "A" 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.

30.3 General Design Details

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

2. Adhesive anchored parapets are allowed at interior Traffic Railing locations only when the adjacent exterior parapet is a crash test approved Traffic Railing per 30.2 (i.e., cast-in-place anchors are used at exterior parapet location). See Standards for Parapet Footing and Lighting Detail for more information.

3. Sign structures, sign trusses, and monotubes shall be placed on top of railings to meet the working width and zone of intrusion dimensions noted in *FDM 11-45 Section 2.3.6.2.2* and *Section 2.3.6.2.3* respectively.

4. It is desirable to avoid attaching noise walls to bridge railings. However, in the event that noise walls are required to be located on bridge railings, compliance with the setback requirements stated in 30.4 and what is required in *FDM 11-45 Sections 2.3.6.2.2* and *2.3.6.2.3* is not required. *Note: WisDOT is currently investigating the future use of noise walls on bridge structures in Wisconsin.*

5. Temporary bridge barriers shall be designed in accordance with *FDM SDD 14b7*. Where temporary bridge barriers are being used for staged construction, the designer should attempt to meet the required offsets so that the barrier does not require anchorage which would necessitate drilling holes in the new deck.

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

7. Refer to Standard for Vertical Face Parapet "A" – for detailing concrete parapet or sidewalk deflection joints. These joints are used based on previous experience with transverse deck cracking beneath the parapet joints.

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

9. For beam guard type "W" railing, locate the expansion splice at a post or on either side of the expansion joint.

10. Sidewalks - If there is a Traffic Railing between the roadway and an at grade sidewalk, and the roadway side of the Traffic Railing is more than 11'-0" from the exterior edge of deck, access must be provided to the at grade sidewalk for the snooper truck to inspect the underside of the bridge. The sidewalk width must be 10'-0" clear between barriers, including fence (i.e., use a straight fence without a bend). For protective

screening, the total height of parapet and fence need not exceed 8'-0". The boom extension on most snooper trucks does not exceed 11'-0" so provision must be made to get the truck closer to the edge.

11. Where Traffic Railing is utilized between the roadway and an at grade sidewalk, early coordination with the roadway designer should occur to provide adequate clearances off of the structure to allow for proper safety hardware placement and sidewalk width. Additional clearance may be required in order to provide a crash cushion or other device to protect vehicles from the blunt end of the interior Traffic Railing off of the structure.

12. On shared-use bridges, fencing height and geometry shall be coordinated with the Region and the DNR (or other agencies) as applicable. Consideration shall be given to bridge use (i.e., multi-use/snowmobile may require vertical and horizontal clearances to allow grooming machine passage) and location (i.e., stream crossing vs. grade separation).

13. Per **LRFD [13.7.1.1]**, the use of raised sidewalks on structures shall be restricted to roadways with a design speed of 45 mph or less. The height of curbs for sidewalks is usually 6 inches. This height is more desirable than higher heights with regards to safety because it is less likely to vault vehicles. However, a raised curb is not considered part of the safety barrier system. On structure rehabilitations, the height of sidewalk may increase up to 8 inches to match the existing sidewalk height at the bridge approaches. Contact the Bureau of Structures Development Section if sidewalk heights in excess of 8 inches are desired. See Standard for Median and Raised Sidewalk Details for typical raised sidewalk detail information.

14. Pedestrian loads, as described in the AASHTO LRFD Bridge Design Specifications, shall be used to not only design the pedestrian railings on the structure, but shall also be used to design stairway railings that are adjacent to the structure and are part of the contract.

30.4 Railing Aesthetics

Railing aesthetics have become a key component to the design and delivery of bridge projects in Wisconsin. WisDOT Regions, local communities and their leaders use rail aesthetics to draw pedestrians to use the walkways on structures. With the increased desire to use, and frequency of use of aesthetics on railings, it has become increasingly important to set policy for railing aesthetics on bridge structures.

Railing aesthetics policies have been around for multiple decades. In the 1989 version of the AASHTO Standard Specifications, generalities were listed for use with designing bridge rails. Statements such as “Use smooth continuous barrier faces on the traffic side” and “Rail ends, posts, and sharp changes in the geometry of the railing shall be avoided to protect traffic from direction collision with the bridge rail ends” were used as policy and engineering judgment was required by each individual designer. This edition of the Standard Specifications aligned with NCHRP Report 350.

Caltrans conducted full-scale crash testing of various textured barriers in 2002. This testing was the first of its kind and produced acceptable railing aesthetics guidelines for single slope barriers for NCHRP Report 350 TL-3 conditions. Some of the allowable aesthetics were: sandblast textures with a maximum relief of 3/8”, geometric patterns inset into the face of the barrier 1” or less and featuring 45° or flatter chamfered or beveled edges, and any pattern or texture with a maximum relief of 2½” located 24” above the base of the barrier. Later in 2002, Harry W. Taylor, the Acting Director of the Office of Safety Design of FHWA, provided a letter to Caltrans stating that their recommendations were acceptable for use on all structure types.

In 2003, WisDOT published a paper titled, “Acceptable Community Sensitive Design Bridge Rails for Low Speed Streets & Highways in Wisconsin”. The goal of this paper was to streamline what railing aesthetics were acceptable for use on structures in Wisconsin. WisDOT policy at that time allowed vertical faced bridge rails in low speed applications to contain aesthetic modifications. For NHS structures, WisDOT allowed various types of texturing and relief based on crash testing and analysis. Ultimately, WisDOT followed many of the same requirements that were deemed acceptable by FHWA based on the Caltrans study in 2002.

NCHRP Report 554 – Aesthetic Concrete Barrier Design – was published in 2006 to (1) assemble a collection of examples of longitudinal traffic barriers exhibiting aesthetic characteristics, (2) develop design guidelines for aesthetic concrete roadway barriers, and (3) develop specific designs for see-through bridge rails. This publication serves as the latest design guide for aesthetic bridge barrier design and all bridge railings on structures in Wisconsin shall comply with the guidance therein.

The aforementioned tests and studies done on aesthetic features will be considered still applicable under MASH barring further tests or studies.

<u>The application of aesthetics on bridge railings on structures in Wisconsin with a design speed exceeding 45 mph shall comply with the following guidance</u>:

1. All Traffic Railings shall meet the crash testing guidelines outlined in 30.1.

2. The top surface of concrete parapets shall be continuous without raised features (pilasters, pedestals, etc.) that potentially serve as snag points for vehicles or blunt ends for impacts. Any raised feature that could serve as a blunt end or snag point shall be placed as follows:

   Minimum of 2'-3" behind the front face toe of the parapet when used with single slope parapets ("32SS", "36SS", "42SS", or "56SS").

   Minimum of 2'-6" behind the front face toe of the parapet when used with sloped face parapets ("LF" or "HF").

   Minimum of 2'-0" behind the front face of the parapet when used with vertical face parapets ("A").

3. Any railing placed on top of a concrete parapet shall be continuous over the full extents of the bridge.

4. Any concrete parapet placed directly on the deck may contain patterns or textures of any shape and length inset into the front face with the exception noted in #5. The maximum pattern or texture recess into the face of the barrier shall be ½". Note that the typical aesthetic form liner patterns shown on the Standard for Formliner Details are not acceptable for use on the front face of vehicle barriers.

   WisDOT highly recommends the use of smooth front faces of Traffic Railings; especially in high speed applications where the aesthetic features will be negligible to the traveling public. In addition to the increased risk of vehicle snagging, aesthetic treatments on the front face of traffic railings are exposed to vehicle impacts, snowplow scrapes, and exposure to deicing chemicals. Due to these increased risks, future maintenance costs will increase.

5. No patterns with a repeating upward sloping edge or rim in the direction of vehicle traffic shall be permitted.

6. Staining should not be applied to the roadway side face of concrete traffic railings.

The application of aesthetics on bridge railings on structures in Wisconsin with a roadway design speed of 45 mph or less shall comply with the following guidance (see Chapter 4 – Aesthetics for CSS funding implications):

1. All Traffic Railings shall meet the crash testing guidelines outlined in 30.1.

2. The top surface of concrete parapets shall be continuous without raised features (pilasters, pedestals, etc.) that potentially serve as snag points for vehicles or blunt ends for impacts. Any raised feature that could serve as a blunt end or snag point shall be placed a minimum of 1'-0" behind the front face toe of the parapet.

3. Any railing placed on top of a concrete parapet shall be continuous over the full extents of the bridge.

4. Any concrete parapet placed directly on the deck or Combination Railing on a raised sidewalk may contain geometric patterns inset into the front face. The maximum recess into the face of the barrier shall be 1" and shall be placed concurrently with a 45° or flatter chamfered or beveled edge. See Standards for Combination Railings Type 'C1-C6' and Combination Railing Details for one example of this type of aesthetic modification.

   WisDOT highly recommends the use of smooth front faces of Traffic Railings and Combination Railings.

5. Any concrete parapet placed directly on the deck or Combination Railing on a raised sidewalk may contain textures of any shape and length inset into the front face. The maximum depth of the texture shall be ½". Note that the typical aesthetic form liner patterns shown in the Standard Detail for Formliner Details are not acceptable for use on the front face of vehicle barriers.

   WisDOT highly recommends the use of smooth front faces of Traffic Railings and Combination Railings.

6. No patterns with a repeating upward sloping edge or rim in the direction of vehicle traffic shall be permitted.

7. Staining should not be applied to the roadway side face of concrete traffic railings. Staining is allowed on concrete surfaces of Combination Railings placed on a raised sidewalk.

30.5 Objects Mounted On Parapets

When light poles are mounted on top of parapets and the design speed exceeds 45 mph, the light pole must be located behind the back edge of the parapet. See Standards for Light Standard and Junction Box For Parapets and Conduit Details and Notes for typical light pole detail and conduit information. The poles should also be placed over the piers unless there is an expansion joint at that location. If an expansion joint is present, place 4 feet away.

See 6.3.3.7 for more information regarding bench mark disks.

30.6 Protective Screening

Protective screening is a special type of fence constructed on the sides of an overpass to discourage and/or prevent people from dropping or throwing objects onto vehicles passing underneath the structure. Protective screening is generally chain link type fencing attached to steel posts mounted on top of a Traffic Railing (part of a Combination Railing) or on a sidewalk surface (Pedestrian Railing). The top of the protective screening may be bent inward toward the roadway, if mounted on a Traffic Railing and on a raised sidewalk, to prevent objects from being thrown off the overpass structure. The top of the protective screening may also be bent inward toward the sidewalk, if mounted directly to the deck when it is protected by a Traffic Railing between the roadway and a sidewalk at grade. Aesthetics are enhanced by using a colored protective screening which can be coordinated with the color of the structure. See Chapter 30 and Chapter 37 Standard Details for protective screening detail information.

Examples of situations that warrant consideration of protective screening are:

1. Location with a history of, or instances of, objects being dropped or thrown from an existing overpass.

2. All new overpasses if there have been instances of objects being dropped or thrown at other existing overpasses in the area.

3. Overpasses near schools, playgrounds, residential areas or any other locations where the overpass may be used by children who are not accompanied by an adult.

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

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

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

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

Protective screening (or Pedestrian Railing) may be required for particular structures based on the safety requirements of the users on the structure and those below. Roadway designers, bridge designers, and project managers should coordinate this need and relay the information to communities involved when aesthetic details are being formalized.

See *FDM 11-35-1.8* for additional guidance pertaining to protective screening usage requirements.

Occasionally, access to light poles behind protective screening is required or the screening may need repair. To gain access, attach fence stretchers to the fencing and remove one vertical wire by threading or cutting. To repair, attach fence stretchers and thread a vertical wire in place of the one removed by either reusing the one in place or using a new one.

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

See 30.3 for additional guidance with regards to snooper truck access, screening height, and straight vs. bent fencing.

30.7 Medians

The typical height of any required median curb is 6 inches. This will prevent normal crossovers and reduce vaulting on low speed roadways without excessive dead load being applied to the superstructure. On structure rehabilitations, the height of median may increase up to 8” to match the existing median at the bridge approaches. Contact the Bureau of Structures Development Section if median heights in excess of 8 inches are desired. See Standard for Median and Raised Sidewalk Details for typical raised median detail information.

30.8 Railing Rehabilitation

The FHWA, in its implementation plan for MASH, requires that bridge railings on the NHS shall meet the requirements of MASH or NCHRP Report 350. In addition, FHWA states that "Agencies are encouraged to upgrade existing highway safety hardware that has not been accepted under MASH or NCHRP Report 350 during reconstruction projects, during 3R (Resurfacing, Restoration, Rehabilitation), or when the railing system is damaged beyond repair".

WisDOT requirements for the treatment of existing railings for various project classifications are outlined in Table 30.8-1:

Project Classification	**Railing Rehabilitation**
Preventative Maintenance✳ (Resurfacing, Restoration) For letting dates after December 31, 2019: The compliance document will be MASH 2016 Edition	Replacement of bridge railing not in compliance with MASH or NCHRP Report 350 is recommended but not required. Existing railings – both in compliance and not in compliance – with MASH, NCHRP Report 350, and NCHRP Report 230 may be altered to improve the performance of the existing railings where it is not feasible to install an approved railing. Coordination with BOS and BPD is required. NHS Structures: It is strongly encouraged that existing railing that does not comply with MASH, NCHRP Report 350, or NCHRP Report 230 be upgraded to comply with MASH or NCHRP Report 350. Non-NHS Structures: It is strongly encouraged that existing railing that does not comply with MASH, NCHRP Report 350, or NCHRP Report 230 be upgraded to comply with MASH or NCHRP Report 350.

3R✳✳ (Resurfacing, Restoration, Rehabilitation) For letting dates after December 31, 2019: The compliance document will be MASH 2016 Edition	If rehabilitation work, as part of the 3R project, is scheduled or performed which does not widen the structure nor affect the existing railing.	Replacement of bridge railing not in compliance with MASH or NCHRP Report 350 is recommended but not required provided the minimum rail height requirement is met. (Minimum rail height shall be 27" for roadway design speed of 45 mph or less and 32" for roadway design speed exceeding 45 mph.) Existing railings – both in compliance and not in compliance – with MASH, NCHRP Report 350, and NCHRP Report 230 may be altered to improve the performance of the existing railings (i.e., raised to meet the minimum rail height requirement) where it is not feasible to install an approved railing. Coordination with BOS and BPD is required. NHS Structures: Existing railing that does not comply with MASH, NCHRP Report 350, or NCHRP Report 230 and does not meet the minimum rail height requirement shall be upgraded to comply with MASH or NCHRP Report 350. Non-NHS Structures: It is strongly encouraged that existing railing that does not comply with MASH, NCHRP Report 350, or NCHRP Report 230 and does not meet the minimum rail height requirement be upgraded to comply with MASH or NCHRP Report 350.
	If rehabilitation work, as part of the 3R project, is scheduled or performed which widens the structure to either side, redecks (full-depth) any complete span of the structure, or if any work affecting the rail is done to the existing structure.	All railing on the structure must comply with MASH or NCHRP Report 350. Limited project by project exceptions may be granted based on coordination and input by the Bureau of Structures and the Wisconsin Division of FHWA Structures Engineer.
4R (Resurfacing, Restoration, Rehabilitation, Reconstruction) For letting dates after December 31, 2019: The compliance document will be MASH 2016 Edition		All railing on the structure must comply with MASH or NCHRP Report 350. Limited project by project exceptions may be granted based on coordination and input by the Bureau of Structures and the Wisconsin Division of FHWA Structures Engineer.

Table 30.8-1
WisDOT Requirements for Retrofitting/Upgrading Bridge Railings to Current Standards

✳ Examples of Preventative Maintenance projects include, but are not limited to:

1. Bridge deck work: Concrete deck repair, patching, and concrete overlays; asphaltic overlays; epoxy and polymer overlays; expansion joint replacement when done in conjunction with an overlay or expansion joint elimination; chloride extraction; installation of a cathodic protection system.

2. Superstructure and substructure work: Steel structure cleaning and repainting, including complete repainting, zone painting, and spot painting with overcoat; structural repairs (except vehicle impact damage); bearing repair or replacement.

∗∗ Examples of 3R projects include, but are not limited to:

1. Bridge deck work: Bridge deck widenings and re-decks; expansion joint replacement when done in conjunction with an overlay or expansion joint elimination; approach slab replacement.

2. Superstructure and substructure work: Wing wall replacement; emergency bridge repair; structural repairs to railings based on vehicle impact damage;

The minimum railing height shall be measured from the top inside face of the railing to the top of the roadway surface at the toe of railing.

For all railing rehabilitations that require upgrades to comply with MASH or NCHRP Report 350, railings shall be employed as discussed in 30.2.

The following is a list of typical railing types that are in service on structures in Wisconsin. The underlined railings comply with MASH, NCHRP Report 350, or NCHRP Report 230 and may remain in service within rehabilitation projects. The *italicized* railings shall be removed from service within rehabilitation projects.

1. Single slope parapet “32SS”, “36SS”, “42SS”, “56SS”. See 30.2.

2. Sloped face parapet "LF". Railing may be used for rehabilitation projects. Meets TL-3 under MASH.

3. Sloped face parapet "HF". Railing may be used for rehabilitation projects. Meets TL-3 under MASH.

4. Vertical face parapet “A”. Railing may be used for rehabilitation projects. See 30.2.

5. Aesthetic parapet “TX”. Railing may be used for rehabilitation projects. Meets TL-2 under MASH.

6. Type “PF” tubular railing. Railing may be used for rehabilitation projects. Meets TL-2 under MASH. Standard Details are in Chapter 40.

7. Type “H” railing. Railing may be used for rehabilitation projects. Meets TL-3 under MASH.

8. Timber Railing. Railing may be used for rehabilitation projects if not on the NHS. Timber railings have not been tested according to MASH.

9. Type "W" railing. Railing may be used for rehabilitation projects on non-NHS structures only. Meets TL-2 under MASH.

10. Type “M” railing. Railing may be used for rehabilitation projects. Meets TL-2 under MASH.

11. Type “NY3/NY4” steel railings. Railing may be used for rehabilitation projects. Meets TL-2 under MASH.

12. *Type “F” railing.* Railing may not be used for rehabilitation projects. Standard Details in Chapter 40 are for informational purposes only.

13. Sloped face parapet “B”. Railing may be used for rehabilitation projects. Meets TL-3 under MASH.

The region shall contact the Bureau of Structures Development Section to determine the sufficiency of existing railings not listed above.

Rehabilitation or improvement projects to historically significant bridges require special attention. Typically, if the original railing is present on a historic bridge, it will likely not meet current crash testing requirements. In some cases, the original railing will not meet current minimum height and opening requirements. There are generally two different options for upgrading railings on historically significant bridges – install a crash-tested Traffic Railing to the interior side of the existing railing and leave the existing railing in place or replace the existing railing with a crash-tested Traffic Railing. Other alternatives may be available but consultation with the Bureau of Structures Development Section is required.

30.9 Railing Guidance for Railroad Structures

Per an April 2013 memorandum written by M. Myint Lwin, Director of the FHWA Office of Bridge Technology, bridge parapets, railings, and fencing shall conform to the following requirements when used in the design and construction of grade separated highway structures over railroads:

1. For NHS bridges over railroad:

   Bridge railings shall comply with AASHTO standards. For Federal-aid highway projects, the designer shall follow normal bridge railing specifications, design standards, and guidelines.

   However, railings for use on NHS bridges over railroads shall be governed by the railroad's standards, regardless of whether the bridge is owned by the railroad or WisDOT. For the case where an NHS bridge crosses over railroads operated by multiple authorities with conflicting parapet, railing, or fencing requirements, standards as agreed by the various railroad authorities and as approved by WisDOT shall be used.

2. For non-NHS bridges over railroad:

   Bridge railings shall comply with the policies outlined within this chapter. For Federal-aid highway projects, the designer shall follow normal bridge railing specifications, design standards, and guidelines.

   All federally funded non-NHS bridges including those over railroads shall be governed by WisDOT's policies outlined above, even if they differ from the railroad's standards.

30.10 References

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

2. American Association of State Highway and Transportation Officials. *Manual for Assessing Safety Hardware*.

3. National Cooperative Highway Research Program. *NCHRP Report 554 – Aesthetic Concrete Barrier Design*.

4. State of California, Department of Transportation. *Crash Testing of Various Textured Barriers*.

5. National Cooperative Highway Research Program. *NCHRP Report 350 – Recommended Procedures for the Safety Performance Evaluation of Highway Features*.

6. State of Wisconsin, Department of Transportation. *Facilities Development Manual*.

7. State of Wisconsin, Department of Transportation. *Wisconsin Bicycle Facility Design Handbook.*

8. State of Wisconsin, Department of Transportation. *Memorandum of Understanding between Wisconsin Department of Transportation, Wisconsin County Highway Association, and Transportation Builders Association.*

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32.1 General

The Regional Office shall determine the utilities that will be affected by the construction of any bridge structure at the earliest possible stage. It shall be their responsibility to deal with these utilities and to provide location plans or any other required sketches for their information. When the utility has to be accommodated on the structure, the Regional Office shall secure approval from the representative of the utility and the Bureau of Structures for the location and method of support.

Due consideration shall be given to the weight of the pipes, ducts, etc. in the design of the beams and diaphragms. To insure that the function, aesthetics, painting and inspection of stringers of a structure are maintained, the following applies to the installation of utilities on structures:

1. Permanent installations, which are to be carried on and parallel to the longitudinal axis of the structure, shall be placed out of sight, between the fascia beams and above the bottom flanges, on the underside of the structure.

2. Conglomeration of utilities in the same bay shall be avoided in order to facilitate maintenance painting and future inspection of girders in a practical manner.

3. In those instances where the proposed type of superstructure is not adaptable to carrying utilities in an out-of-sight location in the underside of the structure, an early determination must be made as to whether or not utilities are to be accommodated and, if so, the type of superstructure must be selected accordingly.

32.6 Conduit Systems

Structures may require conduit systems when lighting, signals and other services are located on or adjacent to structures. These systems typically include conduit, conduit boxes (junction boxes and/or pull boxes), and conduit fittings. Preferably, these conduit system are embedded in concrete elements for protective and aesthetic reasons. In some cases, externally mounted systems may be warranted when concrete embedment is not practical or economical.

Rigid nonmetallic conduit, also referred to as PVC (polyvinyl chloride) conduit, is commonly used throughout structures due to its low costs and ease of installation. At joint locations with fittings, rigid metallic conduit (RMC) is recommended on both sides of the joint for a rigid and durable connection. RMC shall be galvanized per the specifications. Use of reinforced thermosetting resin conduit (RTRC), also referred to as fiberglass conduit, has been limited on projects due to its high costs and durability concerns when embedded in concrete. Use of liquid-tight flexible metal conduit (LFMC) may be considered at large expansion joints or when anticipated movements exceed standard fitting allowances. Use of PVC coated RMC is currently not being used on structures.

For long conduit runs, junction boxes are required to facilitate wire installations, to allow for future access, and for grounding purposes. The maximum run of 2-inch conduit, without a junction box, is 190 feet. Junction boxes can only be used with 2-inch diameter conduit. The maximum run of 3-inch conduit is 190 feet and junction boxes are not allowed to accommodate longer runs. Junction boxes should be used near expansion joints for grounding purposes. Additionally, all expansion fittings are to be wrapped and include a bonding jumper. Pull boxes, similar to junction boxes, are located off of structures and facilitate roadway conduit requirements and details. Typically, these items are addressed in the roadway plans.

See Standards for Light Standard and Junction Box for Parapets and Conduit Details and Notes for additional information. Refer to Chapter 39 for conduit systems servicing sign structures.

Conduit systems for structures should also consider the following items:

- Plans shall specify type, size and location for all conduit, junction boxes, and fittings necessary to accommodate services on structures. Typically, all other electrical requirements such as wiring diagrams, grounding conductors, etc. should be provided in the roadway plans. Additional details and notes may be required for some services, such as conduit systems for navigation lighting.

- Conduit type (coordinate with the Regional electrical engineer):

    - Concrete embedment: 2-inch PVC - schedule 40

    - Concrete embedment at expansion fittings: RMC (3'-0" minimum on each side of fitting)

    - Structure mounted - underdeck lighting: 1-inch RTRC. Refer to Roadway Standards for additional underdeck light details.

  - Structure mounted - Other: 1-inch PVC - schedule 80 (preferred). RTRC, RMC or LFMC may also be considered.

- The maximum allowable conduits that can be placed in concrete parapets is two 3-inch diameter conduits.

- Future conduit runs should not be placed unless future services are highly probable. Conduit systems are expensive and are routinely addressed by maintenance.

- All conduit runs shall have a limited number of bends. The sum of the individual conduit bends on a single run between boxes (pull or junction) shall not exceed 360 degrees, preferably not to exceed 270 degrees. No individual bend shall be greater than 90 degrees. Use two 45 degree bends in lieu of a 90 degree bend when space allows.

- Bends shall not be less than the minimum radius as specified by the National Electrical Code. For layout purposes, all bends shall have a minimum bend radius no less than 6 times the nominal diameter.

- Provide 3'-0" minimum RMC conduit on each side of semi-expansion joint fittings. For expansion joints, provide 3'-0" minimum RMC conduit on one side and extend the other side to a junction box. All semi-expansion and expansion joints with RMC conduit and fittings should be wrapped and bonded, as shown or noted in the Standards.

- For large movements or when joints exceed standard fitting allowances consider using a LFMC system. The specified LFMC conduit length should be at least 2 times the anticipated movements.

- Extend conduit a minimum of 2 inches above concrete surfaces and extend a minimum of 6 inches for buried applications. Provide temporary end caps, unless conduit terminates in a pull box.

- Provide 2'-0" minimum conduit cover when installed below roadways, 1'-6" minimum otherwise. Conduit cover should not exceed 3'-0". Provide 2-inch PVC - schedule 40 for buried applications, unless directed otherwise. Provide 2" minimum concrete cover when embedding in concrete.

- Conduit systems and light spacing requirements should be coordinated with the roadway engineer and the Regional electrical engineer.

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36.1 Design Method

36.1.1 Design Requirements

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

36.1.2 Rating Requirements

The current version of *AASHTO 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.

36.3.3 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 [5.7.3.2.3]** (tension reinforcement only) equals:

$$M_n = A_s f_s \left(d_s - \frac{a}{2}\right)$$

The factored resistance, M_r, or moment capacity per **LRFD [5.7.3.2.1]**, shall be taken as:

$$M_r = \phi M_n = \phi A_s f_s \left(d_s - \frac{a}{2}\right)$$

For additional information on concrete moment capacity, including stress and strain assumptions used, refer to 18.3.3.2.1.

The location of the design moment will consider the haunch dimensions in accordance with **LRFD [12.11.4.2]**. No portion of the haunch shall be considered in adding to the effective depth of the section.

36.3.4.3 Shear Capacity

Per **LRFD [12.11.4.1]**, shear in culverts shall be investigated in conformance with **LRFD [5.14.5.3]**. The location of the critical section for shear for culverts with haunches shall be determined in conformance with **LRFD [C5.13.3.6.1]** and shall be taken at a distance d_v from the end of the haunch.

36.3.4.3.1 Depth of Fill Greater than or Equal to 2.0 ft.

The shear resistance of the concrete, V_c, for slabs of box culverts with 2.0 feet or more of fill, for one-way action per **LRFD [5.14.5.3]** shall be determined as:

$$V_c = \left(0.0676\lambda\sqrt{f'_c} + 4.6\frac{A_s}{bd_e}\frac{V_u d_e}{M_u}\right)bd_e \leq 0.126\lambda\sqrt{f'_c}\,bd_e$$

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)

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)

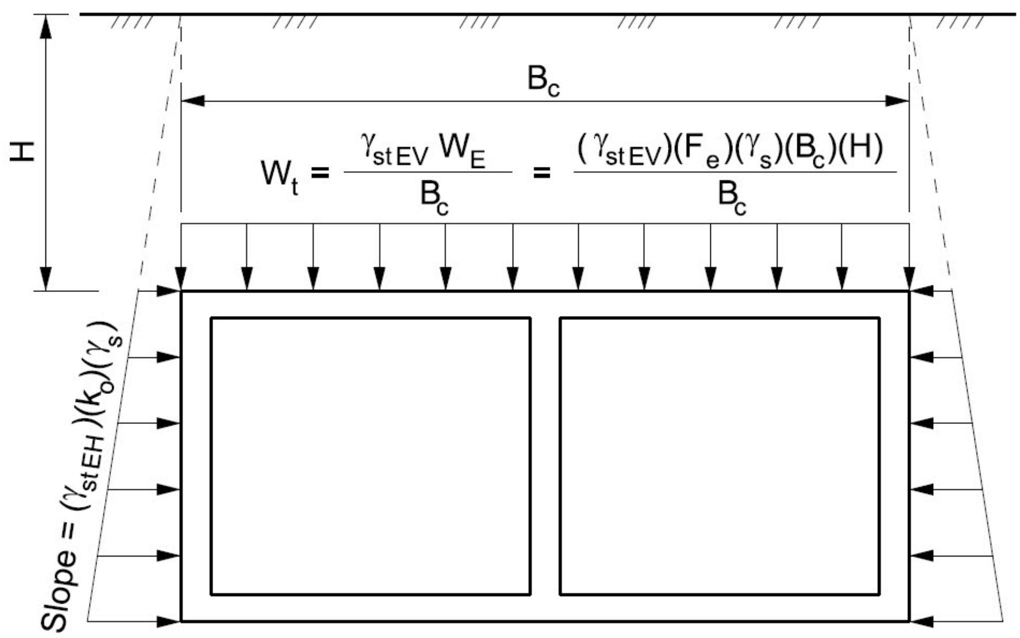

Figure 36.4-1
Factored Vertical and Horizontal Earth Pressures

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.

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E36-1 Twin Cell Box Culvert LRFD

This example shows design calculations for a twin cell box culvert. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. ***(Example is current through LRFD Seventh Edition - 2016 Interim)***

E36-1.1 Design Criteria

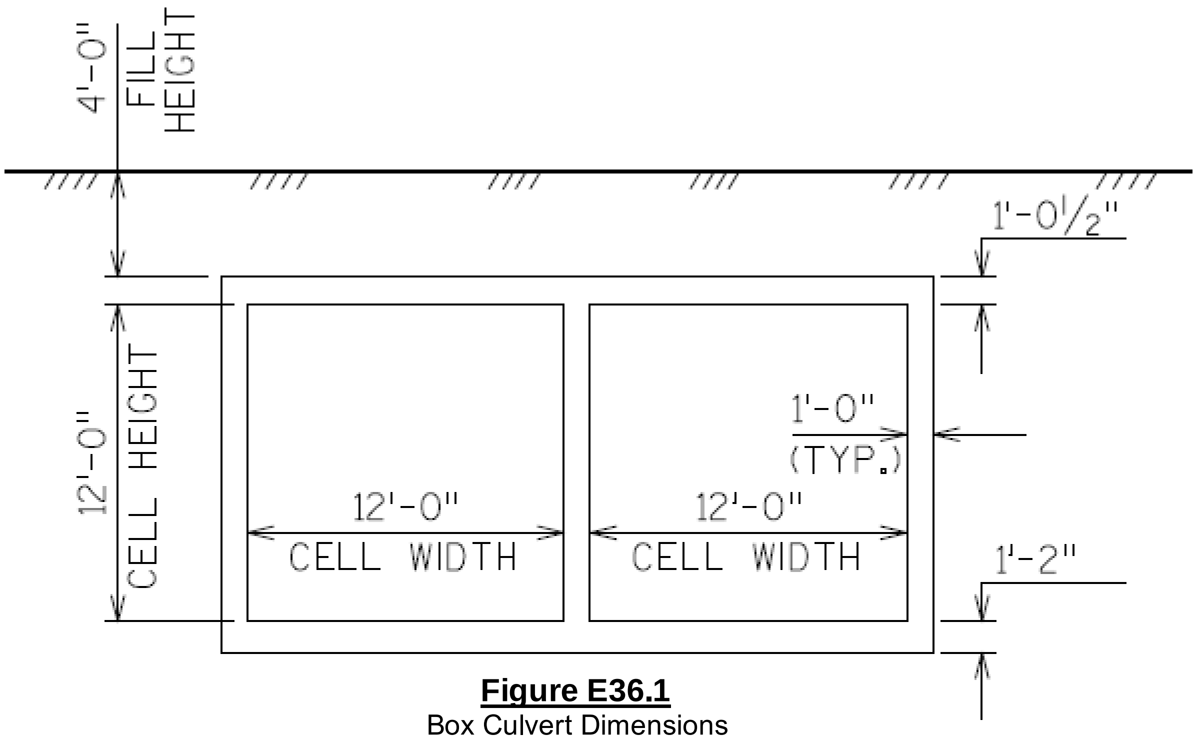


Figure E36.1
Box Culvert Dimensions

$NC = 2$	number of cells
$Ht = 12.0$	cell height, ft
$W_1 = 12.0$	cell 1 width, ft
$W_2 = 12.0$	cell 2 width, ft
$L = 134.0$	culvert length, ft
$t_{ts} = 12.5$	top slab thickness, in
$t_{bs} = 14.0$	bottom slab thickness, in
$t_{win} = 12.0$	interior wall thickness, in
$t_{wex} = 12.0$	exterior wall thickness, in

$$H_{apron} := Ht + \frac{t_{ts}}{12}$$ apron wall height above floor, ft

$H_{apron} = 13.04$ ft.

$f'_c := 3.5$	culvert concrete strength, ksi
$f_y := 60$	reinforcement yield strength, ksi
$E_s := 29000$	modulus of elasticity of steel, ksi
$skew = 0.0$	skew angle, degrees
$H_s = 4.00$	depth of backfill above top edge of top slab, ft
$w_c := 0.150$	weight of concrete, kcf
$cover_{bot} := 3$	concrete cover (bottom of bottom slab), in
$cover := 2$	concrete cover (all other applications), in
$LS_{ht} := 2.2$	live load surcharge height, ft (See Sect. 36.4.4)

Resistance factors, reinforced concrete cast-in-place box structures, **LRFD [Table 12.5.5-1]**

$\phi_f := 0.9$	resistance factor for flexure
$\phi_v := 0.85$	resistance factor for shear

Calculate the span lengths for each cell (measured between centerlines of walls)

$$S_1 := W_1 + \frac{1}{12}\left(\frac{t_{win}}{2} + \frac{t_{wex}}{2}\right) \qquad \boxed{S_1 = 13.00} \text{ ft}$$

$$S_2 = W_2 + \frac{1}{12}\left(\frac{t_{wex}}{2} + \frac{t_{win}}{2}\right) \qquad \boxed{S_2 = 13.00} \text{ ft}$$

Verify that the box culvert dimensions fall within WisDOT's minimum dimension criteria. Per Sect. 36.2, the minimum size for pedestrian underpasses is 8 feet high by 5 feet wide. The minimum size for cattle underpass is 6 feet high by 5 feet wide. A minimum height of 5 feet is desirable for cleanout purposes.

Does the culvert meet the minimum dimension criteria? $\boxed{check = \text{"OK"}}$

Verify that the slab and wall thicknesses fall within WisDOT's minimum dimension criteria. Per Sect. 36.5, the minimum thickness of the top and bottom slab is 6.5 inches. Per Sect. 36.5 [Table 36.5-1], the minimum wall thickness varies with respect to cell height and apron wall height.

Minimum Wall Thickness (Inches)	Cell Height (Feet)	Apron Wall Height Above Floor (Feet)
8	< 6	< 6.75
9	6 to < 10	6.75 to < 10
10	10 to > 10	10 to < 11.75
11		11.75 to < 12.5
12		12.5 to 13

Table 36.5-1
Minimum Wall Thickness Criteria

Do the slab and wall thicknesses meet the minimum dimension criteria? $\boxed{check = \text{"OK"}}$

Since this example has more than 2.0 feet of fill, edge beams are not req'd, **LRFD [C12.11.2.1]**

E36-1.2 Modulus of Elasticity of Concrete Material

Per Sect. 36.2.1, use $f`_c$ = 3.5 ksi for culverts. Calculate value of E_C per **LRFD [C5.4.2.4]**:

$\boxed{K_1 := 1}$ $\quad E_{c\_calc} := 33000 \cdot K_1 \cdot w_c^{1.5} \cdot \sqrt{f'_c}$ $\quad \boxed{E_{c\_calc} = 3586.616}$ ksi

$E_c := 3600$ ksi — modulus of elasticity of concrete, per Sect. 9.2

E36-1.3 Loads

$\gamma_s := 0.120$ — unit weight of soil, kcf

Per Sect. 36.5, a haunch is provided only when the slab depth required at the interior wall is more than 2 inches greater than that required for the remainder of the span. Minimum haunch depth and length is 6 inches. Haunch depth is increased in 3 inch increments. For the first iteration, assume there are no haunches.

$h_{hau} := 0.0$ — haunch height, in

$l_{hau} := 0.0$ — haunch length, in

$wt_{hau} = 0.0$ — weight of one haunch, kip

E36-1.3.1 Dead Loads

Dead Load (DC):

top slab dead load:

$$w_{dlts} := w_c \cdot \frac{t_{ts}}{12} \cdot 1 \qquad \boxed{w_{dlts} = 0.156} \text{ klf}$$

bottom slab dead load:

$$w_{dlbs} := w_c \cdot \frac{t_{bs}}{12} \cdot 1 \qquad \boxed{w_{dlbs} = 0.175} \text{ klf}$$

Wearing Surface (DW):

Per Sect. 36.4.2, the weight of the future wearing surface is zero if there is any fill depth over the culvert. If there is no fill depth over the culvert, the weight of the future wearing surface shall be taken as 0.020 ksf.

$w_{ws} = 0.000$ weight of future wearing surface, ksf

Vertical Earth Load (EV):

Calculate the modification of earth loads for soil-structure interaction per **LRFD [12.11.2.2]**. Per the policy item in Sect. 36.4.3, embankment installations are always assumed.

Installation_Type = "Embankment"

$\gamma_s = 0.120$ unit weight of soil, kcf

$B_c = 27.00$ outside width of culvert, ft
(measured between outside faces of exterior walls)

$H_s = 4.00$ depth of backfill above top edge of top slab, ft

Calculate the soil-structure interaction factor for embankment installations:

$$F_e := 1 + 0.20 \cdot \frac{H_s}{B_c} \qquad \boxed{F_e = 1.03}$$

F_e shall not exceed 1.15 for installations with compacted fill along the sides of the box section:

$$\boxed{F_e = 1.03}$$

Calculate the total unfactored earth load:

$W_E := F_e \cdot \gamma_s \cdot B_c \cdot H_s$ $\boxed{W_E = 13.34}$ klf

Distrubute the total unfactored earth load to be evenly distributed across the top of the culvert:

$w_{sv} := \frac{W_E}{B_c}$ $\boxed{w_{sv} = 0.494}$

Horizontal Earth Load (EH):

Soil horizontal earth load (magnitude at bottom and top of wall): **LRFD [3.11.5.1]**

$k_o := 0.5$ coefficient of at rest lateral earth pressure per Sect. 36.4.3

$\gamma_s = 0.120$ unit weight of soil, kcf

$w_{sh\_bot} := k_o \cdot \gamma_s \cdot \left(Ht + \frac{t_{ts}}{12} + \frac{t_{bs}}{12} + H_s\right) \cdot 1$ $\boxed{w_{sh\_bot} = 1.09}$ klf

$w_{sh\_top} := k_o \cdot \gamma_s \cdot (H_s) \cdot 1$ $\boxed{w_{sh\_top} = 0.24}$ klf

Live Load Surcharge (LS):

Soil live load surcharge: **LRFD [3.11.6.4]**

$k_o = 0.5$ coefficient of lateral earth pressure

$\gamma_s = 0.120$ unit weight of soil, kcf

$LS_{ht} = 2.2$ live load surcharge height per Sect. 36.4.4, ft

$w_{sll} := k_o \cdot \gamma_s \cdot LS_{ht} \cdot 1$ $\boxed{w_{sll} = 0.13}$ klf

E36-1.3.2 Live Loads

For Strength 1 and Service 1:

HL-93 loading = design truck (no lane) **LRFD [3.6.1.3.3]**

design tandem (no lane)

For the Wisconsin Standard Permit Vehicle (Wis-SPV) Check:

The Wis-SPV vehicle is to be checked during the design phase to make sure it can carry a minimum vehicle load of 190 kips. See Section 36.1.3 of the Bridge Manual for requirements pertaining to the Wis-SPV vehicle check.

E36-1.4 Live Load Distribution

Live loads are distributed over an equivalent area, with distribution components both parallel and perpendicular to the span, as calculated below. Per **LRFD [3.6.1.3.3]**, the live loads to be placed on these widths are axle loads (i.e., two lines of wheels) without the lane load. The equivalent distribution width applies for both live load moment and shear.

E36-1.5 Equivalent Strip Widths for Box Culverts

The calculations for depths of fill less than 2.0 ft, per **LRFD [4.6.2.10]** are not required for this example. The calcuations are shown for illustration purposes only.

The calculations below follow **LRFD [4.6.2.10.2]** - Case 1: Traffic Travels Parallel to Span. If traffic travels perpendicular to the span, follow **LRFD [4.6.2.10.3]** - Case 2: Traffic Travels Perpendicular to Span, which states to follow **LRFD [4.6.2.1]**.

Per **LRFD [4.6.2.10.2]**, when traffic travels primarily parallel to the span, culverts shall be analyzed for a single loaded lane with a single lane multiple presence factor (mpf).

Therefore, $mpf = 1.2$

Perpendicular to the span:

It is conservative to use the largest distribution factor from each span of the structure across the entire length of the culvert. Therefore, use the smallest span to calculate the smallest strip width. That strip width will provide the largest distribution factor.

$S := \min(W_1, W_2)$ clear span, ft $\boxed{S = 12.00}$ ft

The equivalent distribution width perpendicular to the span is:

$E_{perp} := \frac{1}{12} \cdot (96 + 1.44 \cdot S)$ $\boxed{E_{perp} = 9.44}$ ft

Parallel to the span:

$H_s = 4.00$ depth of backfill above top edge of top slab, ft

$L_T := 10$ length of tire contact area, in **LRFD [3.6.1.2.5]**

$LLDF = 1.15$ live load distribution factor. From **LRFD [4.6.2.10.2]**, LLDF = 1.15 as specified in **LRFD [Table 3.6.1.2.6a-1]** for select granular backfill

The equivalent distribution width parallel to the span is:

$E_{parallel} := \frac{1}{12} \cdot (L_T + LLDF \cdot H_s \cdot 12)$ $\boxed{E_{parallel} = 5.43}$ ft

The equivalent distribution widths parallel and perpendicular to the span create an area that the axial load shall be distributed over. The equivalent area is:

$E_{area} := E_{perp} \cdot E_{parallel}$ $\boxed{E_{area} = 51.29}$ ft^2

For depths of fill 2.0 ft. or greater calculate the size of the rectangular area that the wheels are considered to be uniformly distributed over, per Sect. 36.4.6.2.

$L_T = 10.0$ length of tire contact area, in **LRFD [3.6.1.2.5]**

$W_T := 20$ width of tire contact area, in **LRFD [3.6.1.2.5]**

The length and width of the equivalent area for 1 wheel are: **LRFD [3.6.1.2.6b]**

$L_{eq\_i} := L_T + LLDF \cdot H_s \cdot 12$ $\boxed{L_{eq\_i} = 65.20}$ in

$W_{eq\_i} := W_T + LLDF \cdot H_s \cdot 12 + 0.06 \cdot \max(W_1, W_2) 12$ $\boxed{W_{eq\_i} = 83.84}$ in

Where such areas from several wheels overlap, the total load shall be uniformly distributed over the area, **LRFD [3.6.1.2.6a]**.

Check if the areas overlap = "Yes, the areas overlap" therefore, use the following length and width values for the equivalent area for 1 wheel:

	Front and Rear Wheels:		Center Wheel:	
Length	$L_{eq13} = 65.2$	in	$L_{eq2} = 65.2$	in
Width	$W_{eq13} = 77.9$	in	$W_{eq2} = 77.9$	in
Area	$A_{eq13} = 5080.4$	in^2	$A_{eq2} = 5080.4$	in^2

Per **LRFD [3.6.1.2.2]**, the weights of the design truck wheels are below. (Note that one axle load is equal to two wheel loads.)

$W_{wheel1i} := 4000$ front wheel weight, lbs

$W_{wheel23i} := 16000$ center and rear wheel weights, lbs

The effect of single and multiple lanes shall be considered. For this problem, a single lane with the single lane multiple presence factor (mpf) governs. Applying the single lane multiple presence factor:

$W_{wheel1} := mpf \cdot W_{wheel1i}$ $W_{wheel1} = 4800.00$ lbs $mpf = 1.20$

$W_{wheel23} := mpf \cdot W_{wheel23i}$ $W_{wheel23} = 19200.00$ lbs

For single-span culverts, the effects of the live load may be neglected where the depth of fill is more than 8.0 feet and exceeds the span length. For multiple span culverts, the effects of the live load may be neglected where the depth of fill exceeds the distance between faces of endwalls, **LRFD [3.6.1.2.6a]**.
Note: The wheel pressure values shown here are for the 14'-0" variable axle spacing of the design truck, which controls over the design tandem for this example. In general, all variable axle spacings of the design truck and the design tandem must be investigated to account for the maximum response. Dividing the wheel loads (incl. mpf) by the equivalent area gives:

$LL1 = 0.94$ live load pressure (front wheel), psi

$LL2 = 3.78$ live load pressure (center wheel), psi

$LL3 = 3.78$ live load pressure (rear wheel), psi

E36-1.6 Limit States and Combinations

The limit states, load factors and load combinations shall be applied as required and detailed in Chapter 36 of this manual and as indicated below.

E36-1.6.1 Load Factors

From **LRFD [Table 3.4.1-1]** and **LRFD [Table 3.4.1-2]**:

Per the policy item in Sect. 36.4.3: Assume box culverts are closed, rigid frames for Strength 1 (EV-factor).

	Strength 1	Service 1
DC	$\gamma st_{DCmax} := 1.25$	$\gamma s1_{DC} := 1.0$
	$\gamma st_{DCmin} := 0.9$	
DW	$\gamma st_{DWmax} := 1.5$	$\gamma s1_{DW} := 1.0$
	$\gamma st_{DWmin} := 0.65$	
EV	$\gamma st_{EVmax} := 1.35$	$\gamma s1_{EV} := 1.0$
	$\gamma st_{EVmin} := 0.9$	
EH	$\gamma st_{EHmax} := 1.35$	$\gamma s1_{EH} := 1.0$
	$\gamma st_{EHmin} := 0.5$ **LRFD [3.11.7]**	
LS	$\gamma st_{LSmax} := 1.75$	$\gamma s1_{LS} := 1.0$
	$\gamma st_{LSmin} := 0$	
LL	$\gamma st_{LL} := 1.75$	$\gamma s1_{LL} := 1.0$

Dynamic Load Allowance (IM) is applied to the truck and tandem. From **LRFD [3.6.2.2]**, IM of buried components varies with depth of cover above the structure and is calculated as:

$IM := 33 \cdot (1.0 - 0.125 \cdot H_s)$ (where H_s is in feet) $IM = 16.50$

If IM is less than 0, use IM = 0 $\boxed{IM = 16.50}$

E36-1.6.2 Dead Load Moments and Shears

The unfactored dead load moments and shears for each component are listed below (values are per 1-foot width and are in kip-ft and kip, respectively):

Exterior Wall Unfactored Dead Load Moments (kip-ft)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	-1.52	-1.44	-5.14	-1.01	0.00
0.1	-1.42	-1.54	-0.12	-0.14	0.00
0.2	-1.31	-1.63	3.53	0.55	0.00
0.3	-1.21	-1.73	5.92	1.04	0.00
0.4	-1.10	-1.82	7.14	1.34	0.00
0.5	-1.00	-1.91	7.30	1.46	0.00
0.6	-0.89	-2.01	6.51	1.38	0.00
0.7	-0.79	-2.10	4.87	1.12	0.00
0.8	-0.68	-2.19	2.49	0.66	0.00
0.9	-0.58	-2.29	-0.54	0.01	0.00
1.0	-0.48	-2.38	-4.11	-0.82	0.00

Interior Wall Unfactored Dead Load Moments (kip-ft)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	0.00	0.00	0.00	0.00	0.00
0.1	0.00	0.00	0.00	0.00	0.00
0.2	0.00	0.00	0.00	0.00	0.00
0.3	0.00	0.00	0.00	0.00	0.00
0.4	0.00	0.00	0.00	0.00	0.00
0.5	0.00	0.00	0.00	0.00	0.00
0.6	0.00	0.00	0.00	0.00	0.00
0.7	0.00	0.00	0.00	0.00	0.00
0.8	0.00	0.00	0.00	0.00	0.00
0.9	0.00	0.00	0.00	0.00	0.00
1.0	0.00	0.00	0.00	0.00	0.00

Top Slab Unfactored Dead Load Moments (kip-ft)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	-0.04	-1.14	-5.47	-1.18	0.00
0.1	0.73	1.45	-4.67	-1.00	0.00
0.2	1.27	3.32	-3.87	-0.83	0.00
0.3	1.60	4.48	-3.07	-0.66	0.00
0.4	1.69	4.93	-2.27	-0.49	0.00
0.5	1.56	4.67	-1.47	-0.32	0.00
0.6	1.21	3.69	-0.67	-0.15	0.00
0.7	0.63	2.01	0.13	0.03	0.00
0.8	-0.18	-0.39	0.93	0.20	0.00
0.9	-1.21	-3.50	1.72	0.37	0.00
1.0	-2.46	-7.32	2.52	0.54	0.00

Bottom Slab Unfactored Dead Load Moments (kip-ft)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	-0.60	-0.17	-7.63	-1.42	0.00
0.1	1.36	2.26	-6.51	-1.21	0.00
0.2	2.76	3.98	-5.39	-1.00	0.00
0.3	3.61	4.99	-4.27	-0.79	0.00
0.4	3.91	5.29	-3.15	-0.59	0.00
0.5	3.65	4.87	-2.03	-0.38	0.00
0.6	2.85	3.75	-0.90	-0.17	0.00
0.7	1.49	1.91	0.22	0.04	0.00
0.8	-0.42	-0.64	1.34	0.25	0.00
0.9	-2.88	-3.90	2.46	0.46	0.00
1.0	-5.89	-7.88	3.58	0.67	0.00

Exterior Wall Unfactored Dead Load Shears (kip)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	0.09	-0.08	4.78	0.73	0.00
0.1	0.09	-0.08	3.60	0.59	0.00
0.2	0.09	-0.08	2.50	0.45	0.00
0.3	0.09	-0.08	1.49	0.30	0.00
0.4	0.09	-0.08	0.56	0.16	0.00
0.5	0.09	-0.08	-0.27	0.01	0.00
0.6	0.09	-0.08	-1.03	-0.13	0.00
0.7	0.09	-0.08	-1.69	-0.27	0.00
0.8	0.09	-0.08	-2.27	-0.42	0.00
0.9	0.09	-0.08	-2.76	-0.56	0.00
1.0	0.09	-0.08	-3.17	-0.71	0.00

Interior Wall Unfactored Dead Load Shears (kip)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	0.00	0.00	0.00	0.00	0.00
0.1	0.00	0.00	0.00	0.00	0.00
0.2	0.00	0.00	0.00	0.00	0.00
0.3	0.00	0.00	0.00	0.00	0.00
0.4	0.00	0.00	0.00	0.00	0.00
0.5	0.00	0.00	0.00	0.00	0.00
0.6	0.00	0.00	0.00	0.00	0.00
0.7	0.00	0.00	0.00	0.00	0.00
0.8	0.00	0.00	0.00	0.00	0.00
0.9	0.00	0.00	0.00	0.00	0.00
1.0	0.00	0.00	0.00	0.00	0.00

Top Slab Unfactored Dead Load Shears (kip)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	0.74	2.45	0.67	0.13	0.00
0.1	0.55	1.86	0.67	0.13	0.00
0.2	0.36	1.26	0.67	0.13	0.00
0.3	0.17	0.67	0.67	0.13	0.00
0.4	-0.01	0.08	0.67	0.13	0.00
0.5	-0.20	-0.52	0.67	0.13	0.00
0.6	-0.39	-1.11	0.67	0.13	0.00
0.7	-0.58	-1.70	0.67	0.13	0.00
0.8	-0.76	-2.30	0.67	0.13	0.00
0.9	-0.95	-2.89	0.67	0.13	0.00
1.0	-1.14	-3.48	0.67	0.13	0.00

Bottom Slab Unfactored Dead Load Shears (kip)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	1.86	2.32	0.94	0.16	0.00
0.1	1.40	1.73	0.94	0.16	0.00
0.2	0.94	1.14	0.94	0.16	0.00
0.3	0.48	0.54	0.94	0.16	0.00
0.4	0.02	-0.05	0.94	0.16	0.00
0.5	-0.44	-0.64	0.94	0.16	0.00
0.6	-0.90	-1.24	0.94	0.16	0.00
0.7	-1.36	-1.83	0.94	0.16	0.00
0.8	-1.82	-2.42	0.94	0.16	0.00
0.9	-2.28	-3.01	0.94	0.16	0.00
1.0	-2.74	-3.61	0.94	0.16	0.00

The DC values are the component dead loads and include the self weight of the culvert and haunch (if applicable).

The DW values are the dead loads from the future wearing surface (DW values occur only if there is no fill on the culvert).

The EV values are the vertical earth loads from the fill on top of the box culvert.

The EH values are the horizontal earth loads from the fill on the sides of the box culvert.

The LS values are the live load surcharge loads (assuming $LS_{ht} = 2.2$ feet of surcharge)

E36-1.6.3 Live Load Moments and Shears

The unfactored live load load moments and shears (per lane including impact) are listed below (values are in kip-ft and kips, respectively). A separate analysis run will be required if results without impact are desired.

Exterior Wall Unfactored Live Load Moments (kip-ft)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.73	-1.74	0.74	-1.77
0.1	0.67	-1.70	0.69	-1.92
0.2	0.61	-1.67	0.65	-2.07
0.3	0.55	-1.65	0.62	-2.21
0.4	0.48	-1.68	0.60	-2.36
0.5	0.42	-1.82	0.58	-2.51
0.6	0.37	-1.97	0.56	-2.69
0.7	0.41	-2.12	0.56	-2.86
0.8	0.47	-2.28	0.61	-3.04
0.9	0.55	-2.44	0.68	-3.21
1.0	0.65	-2.61	0.77	-3.39

Interior Wall Unfactored Live Load Moments (kip-ft)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.99	-0.99	0.88	-0.88
0.1	0.93	-0.93	0.99	-0.99
0.2	0.92	-0.92	1.12	-1.12
0.3	0.90	-0.90	1.25	-1.25
0.4	0.90	-0.90	1.38	-1.38
0.5	1.08	-1.08	1.54	-1.53
0.6	1.27	-1.27	1.74	-1.74
0.7	1.47	-1.47	1.99	-1.99
0.8	1.69	-1.69	2.24	-2.24
0.9	1.92	-1.92	2.50	-2.50
1.0	2.17	-2.17	2.75	-2.75

Top Slab Unfactored Live Load Moments (kip-ft)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.81	-1.76	0.65	-2.16
0.1	2.24	-0.34	1.83	-0.20
0.2	3.81	-0.27	4.23	-0.32
0.3	5.06	-0.49	5.92	-0.66
0.4	5.71	-0.75	6.78	-1.04
0.5	5.76	-1.04	6.90	-1.43
0.6	5.22	-1.34	6.21	-1.82
0.7	4.13	-1.64	4.74	-2.22
0.8	2.56	-1.96	2.54	-2.62
0.9	0.86	-3.59	0.76	-3.02
1.0	0.07	-5.89	0.06	-4.81

Bottom Slab Unfactored Live Load Moments (kip-ft)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.46	-0.67	0.40	-0.35
0.1	1.72	-0.29	2.52	-0.32
0.2	3.30	-0.76	4.46	-0.78
0.3	4.25	-1.06	5.63	-1.09
0.4	4.60	-1.24	6.06	-1.30
0.5	4.39	-1.34	5.82	-1.45
0.6	3.68	-1.39	4.96	-1.62
0.7	2.56	-1.46	3.55	-1.86
0.8	1.18	-1.57	1.62	-2.23
0.9	0.00	-2.40	0.00	-2.79
1.0	0.00	-4.90	0.00	-3.75

Exterior Wall Unfactored Live Load Shears (kip)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.11	-0.19	0.09	-0.16
0.1	0.11	-0.19	0.09	-0.16
0.2	0.11	-0.19	0.09	-0.16
0.3	0.11	-0.19	0.09	-0.16
0.4	0.11	-0.19	0.09	-0.16
0.5	0.11	-0.19	0.09	-0.16
0.6	0.11	-0.19	0.09	-0.16
0.7	0.11	-0.19	0.09	-0.16
0.8	0.11	-0.19	0.09	-0.16
0.9	0.11	-0.19	0.09	-0.16
1.0	0.11	-0.19	0.09	-0.16

Interior Wall Unfactored Live Load Shears (kip)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.23	-0.23	0.21	-0.21
0.1	0.23	-0.23	0.21	-0.21
0.2	0.23	-0.23	0.21	-0.21
0.3	0.23	-0.23	0.21	-0.21
0.4	0.23	-0.23	0.21	-0.21
0.5	0.23	-0.23	0.21	-0.21
0.6	0.23	-0.23	0.21	-0.21
0.7	0.23	-0.23	0.21	-0.21
0.8	0.23	-0.23	0.21	-0.21
0.9	0.23	-0.23	0.21	-0.21
1.0	0.23	-0.23	0.21	-0.21

Top Slab Unfactored Live Load Shears (kip)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	2.71	-0.26	3.24	-0.33
0.1	2.33	-0.33	2.67	-0.33
0.2	1.95	-0.47	2.11	-0.33
0.3	1.56	-0.69	1.59	-0.39
0.4	1.19	-1.00	1.14	-0.67
0.5	0.85	-1.37	0.78	-1.03
0.6	0.54	-1.74	0.49	-1.46
0.7	0.30	-2.10	0.27	-1.97
0.8	0.14	-2.44	0.12	-2.54
0.9	0.04	-2.76	0.04	-3.11
1.0	0.00	-3.05	0.00	-3.66

Bottom Slab Unfactored Live Load Shears (kip)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	2.19	-0.68	2.69	-0.68
0.1	1.61	-0.48	1.97	-0.48
0.2	1.06	-0.32	1.29	-0.32
0.3	0.54	-0.19	0.66	-0.21
0.4	0.06	-0.11	0.07	-0.14
0.5	0.01	-0.45	0.00	-0.46
0.6	0.02	-0.90	0.02	-0.96
0.7	0.02	-1.33	0.02	-1.40
0.8	0.01	-1.74	0.01	-1.80
0.9	0.00	-2.12	0.00	-2.15
1.0	0.00	-2.48	0.00	-2.46

E36-1.6.4 Factored Moments

WisDOT's policy is to set all of the load modifiers, η, equal to 1.0. The factored moments for each limit state are calculated by applying the appropriate load factors to loads on a 1-foot strip width of the box culvert. The minimum or maximum load factors may be used on each component to maximize the load effects. The results are as follows:

Strength 1 Moments

$$M_{str1} = \eta \cdot \left(\gamma st_{DC} \cdot M_{DC} + \gamma st_{DW} \cdot M_{DW} + \gamma st_{EV} \cdot M_{EV} + \gamma st_{EH} \cdot M_{EH} + \gamma st_{LS} \cdot M_{LS} + \gamma st_{LL} \cdot M_{LL}\right)$$

Corner Bars	$Mstr1_{CB} = 16.73$	kip-ft	(negative moment)
Positive Moment Top Slab Bars	$Mstr1_{PTS} = 19.59$	kip-ft	(positive moment)
Positive Moment Bottom Slab Bars	$Mstr1_{PBS} = 21.05$	kip-ft	(positive moment)
Negative Moment Top Slab Bars	$Mstr1_{NTS} = 22.00$	kip-ft	(negative moment)
Negative Moment Bottom Slab Bars	$Mstr1_{NBS} = 24.77$	kip-ft	(negative moment)
Exterior Wall Bars	$Mstr1_{XW} = 10.81$	kip-ft	(positive moment)
Interior Wall Bars	$Mstr1_{IW} = 4.82$	kip-ft	(positive moment)

Service 1 Moments

$$M_{s1} = \eta \cdot \left(\gamma s1_{DC} \cdot M_{DC} + \gamma s1_{DW} \cdot M_{DW} + \gamma s1_{EV} \cdot M_{EV} + \gamma s1_{EH} \cdot M_{EH} + \gamma s1_{LS} \cdot M_{LS} + \gamma s1_{LL} \cdot M_{LL}\right)$$

Corner Bars	$Ms1_{CB} = 11.18$	kip-ft	(negative moment)
Positive Moment Top Slab Bars	$Ms1_{PTS} = 11.66$	kip-ft	(positive moment)
Positive Moment Bottom Slab Bars	$Ms1_{PBS} = 12.32$	kip-ft	(positive moment)
Negative Moment Top Slab Bars	$Ms1_{NTS} = 13.15$	kip-ft	(negative moment)
Negative Moment Bottom Slab Bars	$Ms1_{NBS} = 15.08$	kip-ft	(negative moment)
Exterior Wall Bars	$Ms1_{XW} = 6.43$	kip-ft	(positive moment)
Interior Wall Bars	$Ms1_{IW} = 2.75$	kip-ft	(positive moment)

E36-1.7 Design Reinforcement Bars

Design of the corner bars is illustrated below. Calculations for bars in other locations are similar.

Design Criteria:

For corner bars, use the controlling thickness between the slab and wall. The height of the concrete design section is:

$h := \min(t_{ts}, t_{bs}, t_{wex})$ $\boxed{h = 12.00}$ in

Use a 1'-0" design width:

$b := 12.0$ width of the concrete design section, in

$cover = 2.0$ concrete cover, in Note: The calculations here use 2" cover for the top slab and walls. Use 3" cover for the bottom of the bottom slab (not shown here).

$Mstr1_{CB} = 16.73$ design strength moment, kip-ft

$Ms1_{CB} = 11.18$ design service moment, kip-ft

$f_s := f_y$ reinforcement yield strength, ksi $f_y = 60.00$ ksi

$Bar_{No} := 5$ assume #5 bars (for d_s calculation)

$Bar_D(Bar_{No}) = 0.63$ bar diameter, in

Calculate the estimated distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement. **LRFD [5.7.3.2.2]**

$$d_{s\_i} := h - cover - \frac{Bar_D(Bar_{No})}{2} \qquad \boxed{d_{s\_i} = 9.69} \text{ in}$$

For reinforced concrete cast-in-place box structures, $\phi_f = 0.90$ per **LRFD [Table 12.5.5-1]**.

Calculate the coefficient of resistance:

$$R_n := \frac{Mstr1_{CB} \cdot 12}{\phi_f \cdot b \cdot {d_{s\_i}}^2} \qquad \boxed{R_n = 0.20} \text{ ksi}$$

Calculate the reinforcement ratio:

$$\rho := 0.85 \cdot \frac{f'_c}{f_y} \cdot \left(1 - \sqrt{1.0 - \frac{2 \cdot R_n}{0.85 \cdot f'_c}}\right) \qquad \boxed{\rho = 0.0034}$$

Calculate the required area of steel:

$A_{s\_req'd} := \rho \cdot b \cdot d_{s\_i}$ $\boxed{A_{s\_req'd} = 0.40}$ in^2

Given the required area of steel of $A_{s\_req'd} = 0.40$, try #5 bars at 7.5" spacing:

$Bar_{No} := 5$ bar size

$spacing := 7.0$ bar spacing, in

The area of one reinforcing bar is:

$A_{s\_1bar} := Bar_A(Bar_{No})$ $\boxed{A_{s\_1bar} = 0.31}$ in^2

Calculate the area of steel in a 1'-0" width

$$A_s := \frac{A_{s\_1bar}}{\frac{spacing}{12}}$$ $\boxed{A_s = 0.53}$ in^2

Check that the area of steel provided is larger than the required area of steel

Is $A_s = 0.53\ in^2 \geq A_{s\_req'd} = 0.40\ in^2$ $\boxed{check = "OK"}$

Recalculate d_c and d_s based on the actual bar size used.

$$d_c := cover + \frac{Bar_D(Bar_{No})}{2}$$ $\boxed{d_c = 2.31}$ in

$$d_s := h - cover - \frac{Bar_D(Bar_{No})}{2}$$ $\boxed{d_s = 9.69}$ in

Per **LRFD [5.7.2.2]**, The factor β_1 shall be taken as 0.85 for concrete strengths not exceeding 4.0 ksi. For concrete strengths exceeding 4.0 ksi, β_1 shall be reduced at a rate of 0.05 for each 1.0 ksi of strength in excess of 4.0 ksi, except that β_1 shall not be taken to be less than 0.65. The factor α_1 shall be taken as 0.85 for concrete strength not exceeding 10.0 ksi.

$\boxed{\beta_1 = 0.85}$ $\boxed{\alpha_1 = 0.85}$

Per **LRFD [5.7.2.1]**, if $\frac{c}{d_s} \leq 0.6$ (for f_y = 60 ksi) then reinforcement has yielded and the assumption is correct.

"c" is defined as the distance between the neutral axis and the compression face (inches).

$$c := \frac{A_s \cdot f_s}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b}$$ $\boxed{c = 1.05}$ in

Check that the reinforcement will yield:

Is $\frac{c}{d_s} = 0.11 \leq 0.6$? $\boxed{check = "OK"}$

therefore, the reinforcement will yield

Calculate the nominal moment capacity of the rectangular section in accordance with **LRFD [5.7.3.2.3]**:

$a := \beta_1 \cdot c$ $\boxed{a = 0.89}$ in

$M_n := \left[A_s \cdot f_s \cdot \left(d_s - \frac{a}{2}\right)\frac{1}{12}\right]$ $\boxed{M_n = 24.6}$ kip-ft

For reinforced concrete cast-in-place box structures, $\phi_f = 0.90$ **LRFD [Table 12.5.5-1]**. Therefore the usable capacity is:

$M_r := \phi_f \cdot M_n$ $\boxed{M_r = 22.1}$ kip-ft

The required capacity:

Corner Moment $\boxed{Mstr1_{CB} = 16.7}$ kip-ft

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

$b = 12.0$ in width of the concrete design section, in

$h = 12.0$ in height of the concrete design section, in

$f_r = 0.24 \cdot \lambda \sqrt{f'_c}$ = 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]** $f_r = 0.45$ ksi

$I_g := \frac{1}{12} \cdot b \cdot h^3$ gross moment of inertia, in^4 $I_g = 1728.00$ in^4

$\frac{h}{2} = 6.0$ distance from the neutral axis to the extreme element

$S_c := \frac{I_g}{\frac{h}{2}}$ section modulus, in^3 $S_c = 288.00$ in^3

The corresponding cracking moment is:

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

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

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

$M_{cr} := 1.1 f_r \cdot S_c \cdot \frac{1}{12}$ $\boxed{M_{cr} = 11.9}$ kip-ft

$\boxed{1.33 \cdot Mstr1_{CB} = 22.2}$ kip-ft

Is $M_r = 22.1$ kip-ft greater than the lesser of M_{cr} and $1.33{*}M_{str}$? $\boxed{check = \text{"OK"}}$

Per **LRFD [5.7.3.4]**, the spacing(s) of reinforcement in the layer closest to the tension face shall satisfy:

$$s \le \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c \qquad \text{in which:} \quad \beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)}$$

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

$h = 12.0$ height of the concrete design section, in

Calculate the ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face:

$$\beta_s := 1 + \frac{d_c}{0.7 \cdot (h - d_c)} \qquad \boxed{\beta_s = 1.34}$$

Calculate the reinforcement ratio:

$$\rho := \frac{A_s}{b \cdot d_s} \qquad \boxed{\rho = 0.0046}$$

Calculate the modular ratio:

$$N := \frac{E_s}{E_c} \qquad \boxed{N = 8.06}$$

Calculate f_{ss}, the tensile stress in steel reinforcement at the Service I Limit State (ksi). The moment arm used in the equation below to calculate f_{ss} is: (j) (h-d_c)

$$k := \sqrt{(\rho \cdot N)^2 + (2 \cdot \rho \cdot N)} - \rho \cdot N \qquad \boxed{k = 0.2370}$$

$$j := 1 - \frac{k}{3} \qquad \boxed{j = 0.9210}$$

$Ms1_{CB} = 11.18$ service moment, kip-ft

$$f_{ss} := \frac{Ms1_{CB} \cdot 12}{A_s \cdot (j) \cdot (h - d_c)} \le 0.6\, f_y \qquad \boxed{f_{ss} = 28.29} \text{ ksi} \quad \le 0.6\, f_y \text{ O.K.}$$

Calculate the maximum spacing requirements per **LRFD [5.10.3.2]**:

$$s_{max1} := \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c \qquad s_{max1} = 13.83 \quad in$$

$$s_{max2} := \min(1.5 \cdot h, 18) \qquad s_{max2} = 18.00 \quad in$$

$$s_{max} := \min(s_{max1}, s_{max2}) \qquad \boxed{s_{max} = 13.83} \quad in$$

Check that the provided spacing is less than the maximum allowable spacing

Is $spacing = 7.00$ in $\leq$ $s_{max} = 13.83$ in $\boxed{check = \text{"OK"}}$

Calculate the minimum spacing requirements per **LRFD [5.10.3.1]**. The clear distance between parallel bars in a layer shall not be less than:

$$S_{min1} := 1.5 \cdot Bar_D(Bar_{No}) \qquad S_{min1} = 0.94 \quad in$$

$$S_{min2} := 1.5 \cdot 1.5 \quad \text{(maximum aggregate size = 1.5 inches)} \qquad S_{min2} = 2.25 \quad in$$

$$S_{min3} := 1.5 \quad in$$

Is $spacing = 7.00$ in $\geq$ all minimum spacing requirements? $\boxed{check = \text{"OK"}}$

E36-1.8 Shrinkage and Temperature Reinforcement Check

Check shrinkage and temperature reinforcement criteria for the reinforcement selected in preceding sections.

The area of reinforcement (A_s) per foot, for shrinkage and temperature effects, on each face and in each direction shall satisfy: **LRFD [5.10.8]**

$$A_s \geq \frac{1.30 \cdot b \cdot (h)}{2 \cdot (b + h) \cdot f_y} \qquad \text{and} \qquad 0.11 \leq A_s \leq 0.60$$

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

Check the minimum required temperature and shrinkage reinforcement, #4 bars at 15", in the thickest section. For the given cross section, the values for the corner bar design are:

$$A_{s\_4\_at\_15} := \frac{Bar_A(4)}{1.25} \qquad \boxed{A_{s\_4\_at\_15} = 0.16} \quad \frac{in^2}{ft}$$

$b_{TS} := \max(t_{ts}, t_{bs}, t_{wex})$ $\boxed{b_{TS} = 14.0}$ in

$h_{TS} := 12(W_1 + W_2) + 2 \cdot t_{wex} + t_{win}$ $\boxed{h_{TS} = 324.0}$ in

$f_y = 60.00$ ksi

For each face, the required area of steel is:

$$A_{s\_TS} := \frac{1.30 \cdot (b_{TS}) \cdot h_{TS}}{2 \cdot (b_{TS} + h_{TS}) \cdot f_y} \qquad A_{s\_TS} = 0.15 \quad \frac{in^2}{ft}$$

is $A_{s\_4\_at\_15} = 0.16$ in2 $\geq$ $A_{s\_TS} = 0.15$ in^2 ? $\boxed{check = "OK"}$

is $0.11 < A_{s\_4\_at\_15} < 0.60$? $\boxed{check = "OK"}$

Per **LRFD [5.10.8]**, the shrinkage and temperature reinforcement shall not be spaced farther apart than:

- 3.0 times the component thickness, or 18.0 in.
- 12.0 in for walls and footings greater than 18.0 in. thick
- 12.0 in for other components greater than 36.0 in. thick

$s_{max3} = 18.00$ in

Per **LRFD [5.10.3.2]**, the maximum center to center spacing of adjacent bars shall not exceed 1.5 times the thickness of the member or 18.0 in.

$s_{max4} = 18.00$ in

is the 15" spacing $\leq$ both maximum spacing requirements? $\boxed{check = "OK"}$

Note: The design of the bottom slab shrinkage and temperature bars is illistrated above. Shrinkage and temperature bars may be reduced or not required at other locations. See Section 36.6.8 and Standard 36.03 for additional information.

The results for the other bar locations are shown in the table below:

Results						
Location	ΦMn	$A_{S\ Req'd}$	$A_{S\ Actual}$	Bar Size	S_{max}	S_{actual}
Corner	22.1	0.48	0.53	5	13.8	7.0
Pos. Mom. Top Slab	21.8	0.49	0.50	5	13.0	7.5
Pos. Mom. Bot. Slab	28.9	0.54	0.57	5	18.0	6.5
Neg. Mom. Top Slab	23.3	0.50	0.53	5	12.1	7.0
Neg. Mom. Bot. Slab	28.4	0.54	0.62	5	13.4	6.0
Exterior Wall	16.9	0.34	0.40	4	18.0	6.0
Interior Wall	6.9	0.15	0.16	4	18.0	15.0

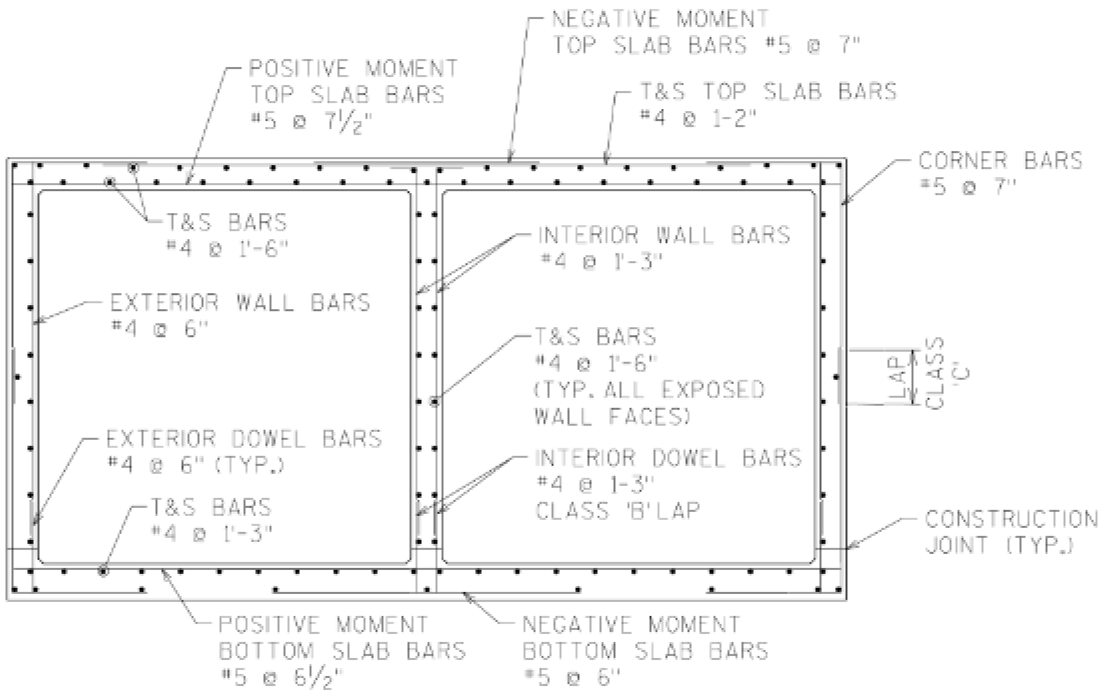
E36-1.9 Distribution Reinforcement

Per **LRFD [9.7.3.2]**, reinforcement shall be placed in the secondary direction in the bottom of slabs as a percentage of the primary reinforcement for positive moment as follows:

Distribution steel is not required when the depth of fill over the slab exceeds 2 feet, **LRFD [5.14.4.1]**.

E36-1.10 Reinforcement Details

The reinforcement bar size and spacing required from the strength and serviceability calcuations above are shown below:



E36-1.11 Cutoff Locations

Determine the cutoff locations for the corner bars. Per Sect. 36.6.1, the distance "L" is computed from the maximum negative moment envelope for the top slab.

The cutoff lengths are in feet, measured from the inside face of the exterior wall.

Initial Cutoff Locations:

The initial cutoff locations are determined from the inflection points of the moment diagrams.

Corner Bars	$CutOff1_{CBH\_i} = 2.64$	$CutOff2_{CBH\_i} = 1.15$	Horizontal
		$CutOff2_{CBV\_i} = 2.07$	Vertical
Positive Moment Top Slab Bars	$CutOff1_{PTS\_i} = 1.26$	$CutOff2_{PTS\_i} = 1.86$	
Positive Moment Bottom Slab Bars	$CutOff1_{PBS\_i} = 1.27$	$CutOff2_{PBS\_i} = 1.97$	
Negative Moment Top Slab Bars	$CutOff1_{NTS\_i} = 8.63$	$CutOff2_{NTS\_i} = 10.32$	
Negative Moment Bottom Slab Bars	$CutOff1_{NBS\_i} = 8.97$	$CutOff2_{NBS\_i} = 10.56$	

For the second cutoff location for each component, the following checks shall be completed:

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

The required capacity at the second cutoff location (for the vertical leg of the corner bar):

$Mstr1_{CBV2} = 7.89$ strength moment at the second cutoff location, kip-ft

The usable capacity of the remaining bars is calculated as follows:

$$A_{s2} := \frac{A_s}{2} \qquad \boxed{A_{s2} = 0.27} \ \text{in}^2$$

$$c2 := \frac{A_{s2} \cdot f_s}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b} \qquad \boxed{\beta_1 = 0.85} \ \boxed{\alpha_1 = 0.85} \qquad \boxed{c2 = 0.53} \ \text{in}$$

$$a2 := \beta_1 \cdot c2 \qquad \boxed{a2 = 0.45} \ \text{in}$$

$$M_{n2} := \left[A_{s2} \cdot f_s \cdot \left(d_s - \frac{a2}{2}\right) \frac{1}{12}\right] \qquad \boxed{M_{n2} = 12.6} \ \text{kip-ft}$$

$$M_{r2} := \phi_f \cdot M_{n2} \qquad \boxed{M_{r2} = 11.3} \ \text{kip-ft}$$

Is $M_{r2} = 11.3$ kip-ft greater than the lesser of M_{cr} and 1.33*M_{str}? $\boxed{check = \text{"OK"}}$

$\boxed{M_{cr} = 11.9}$ kip-ft

$\boxed{1.33 \cdot Mstr1_{CBV2} = 10.5}$ kip-ft

Calculate f_{ss}, the tensile stress in steel reinforcement at the Service I Limit State (ksi).

$Ms1_{CBV2} = 3.43$ service moment at the second cutoff location, kip-ft

$$f_{ss2} := \frac{Ms1_{CBV2} \cdot 12}{A_{s2} \cdot (j) \cdot (h - d_c)}$$ $\boxed{f_{ss2} = 17.35}$ ksi

Calculate the maximum spacing requirements per **LRFD [5.10.3.2]**:

$$s_{max2\_1} := \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss2}} - 2 \cdot d_c$$ $s_{max2\_1} = 25.47$ in

$s_{max2\_2} := s_{max2}$ $s_{max2\_2} = 18.00$ in

$s_{max} := \min(s_{max2\_1}, s_{max2\_2})$ $\boxed{s_{max} = 18.00}$ in

Check that the provided spacing (for half of the bars) is less than the maximum allowable spacing

$spacing2 := 2 \cdot spacing$ $spacing2 = 14.00$ in

Is $spacing2 = 14.00$ in $\leq$ $s_{max} = 18.00$ in $\boxed{check = \text{"OK"}}$

Extension Lengths:

The extension lengths for the corner bars are shown below. Calculations for other bars are similar.

Extension lengths for general reinforcement per **LRFD [5.11.1.2.1]**:

$$MaxDepth := \max\left(t_{ts} - cover, t_{wex} - cover, t_{bs} - cover_{bot}\right) \qquad MaxDepth = 11.00 \quad in$$

Effective member depth $\dfrac{MaxDepth - \frac{1}{2} Bar_D\left(Bar_{No\_CB}\right)}{12} = 0.89$ ft

15 x bar diameter $\dfrac{15 \cdot Bar_D\left(Bar_{No\_CB}\right)}{12} = 0.78$ ft

1/20 times clear span $\dfrac{\max\left(W_1, W_2\right)}{20} = 0.60$ ft

The maximum of the values listed above:

$$ExtendLength\_gen_{CB} = 0.89 \quad ft$$

Extension lengths for negative moment reinforcement per **LRFD [5.11.1.2.3]**:

Effective member depth $\dfrac{MaxDepth - \frac{1}{2} Bar_D\left(Bar_{No\_CB}\right)}{12} = 0.89$ ft

12 x bar diameter $\dfrac{12 \cdot Bar_D\left(Bar_{No\_CB}\right)}{12} = 0.63$ ft

0.0625 times clear span $0.0625 \max\left(W_1, W_2\right) = 0.75$ ft

The maximum of the values listed above:

$$ExtendLength\_neg_{CB} = 0.89 \quad ft$$

The development length:

$$DevLength_{CB} = 1.00 \quad ft$$

The extension lengths for general reinforcment for the other bars are:

Corner Bars	$ExtendLength\_gen_{CB} = 0.89$	ft
Positive Moment Top Slab Bars	$ExtendLength\_gen_{PTS} = 0.85$	ft
Positive Moment Bottom Slab Bars	$ExtendLength\_gen_{PBS} = 0.97$	ft
Negative Moment Top Slab Bars	$ExtendLength\_gen_{NTS} = 0.85$	ft
Negative Moment Bottom Slab Bars	$ExtendLength\_gen_{NBS} = 0.97$	ft

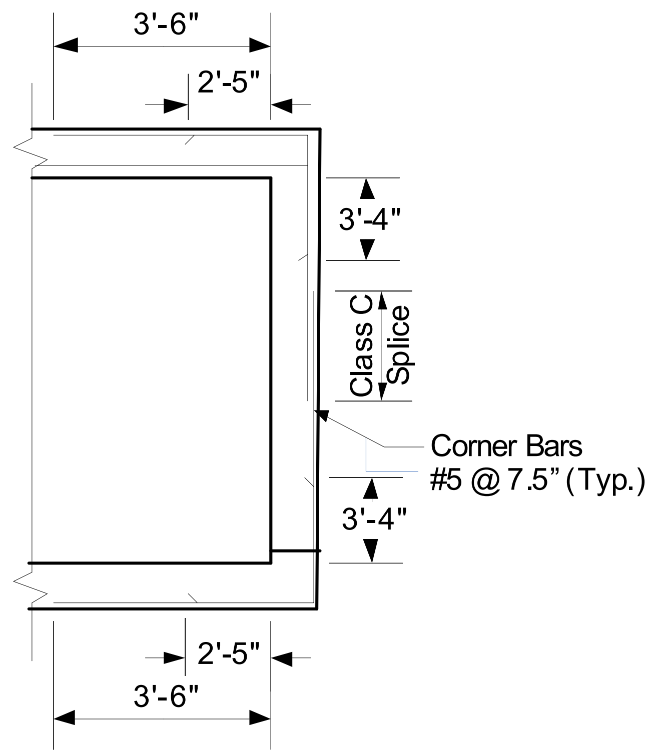
The extension lengths for negative moment reinforcment for the other bars are:

Corner Bars	$ExtendLength\_neg_{CB} = 0.89$	ft
Positive Moment Top Slab Bars	$ExtendLength\_neg_{PTS} = 0.85$	ft
Positive Moment Bottom Slab Bars	$ExtendLength\_neg_{PBS} = 0.97$	ft
Negative Moment Top Slab Bars	$ExtendLength\_neg_{NTS} = 0.85$	ft
Negative Moment Bottom Slab Bars	$ExtendLength\_neg_{NBS} = 0.97$	ft

The final cutoff locations (measured from the inside face of the exterior wall) are:

Corner Bars	$CutOff1_{CBH} = 3.53$	$CutOff2_{CBH} = 2.04$	Horizontal
		$CutOff2_{CBV} = 2.96$	Vertical
Positive Moment Top Slab Bars	$CutOff1_{PTS}$ = "Run Bar Entire Width of Box"		
		$CutOff2_{PTS} = 1.02$	
Positive Moment Bottom Slab Bars	$CutOff1_{PBS}$ = "Run Bar Entire Width of Box"		
		$CutOff2_{PBS} = 1.00$	
Negative Moment Top Slab Bars	$CutOff1_{NTS} = 7.78$	$CutOff2_{NTS} = 9.47$	
Negative Moment Bottom Slab Bars	$CutOff1_{NBS} = 7.99$	$CutOff2_{NBS} = 9.59$	

The cutoff locations for the corner bars are shown below. Other bars are similar.



E36-1.12 Shear Analysis

Analyze walls and slabs for shear

E36-1.12.1 Factored Shears

WisDOT's policy is to set all of the load modifiers, η, equal to 1.0. The factored shears for each limit state are calculated by applying the appropriate load factors to loads on a 1-foot strip width of the box culvert. The minimum or maximum load factors may be used on each component to maximize the load effects. The results are as follows:

<u>Strength 1 Shears</u>

$$V_{str1} = \eta \cdot \left(\gamma st_{DC} \cdot V_{DC} + \gamma st_{DW} \cdot V_{DW} + \gamma st_{EV} \cdot V_{EV} + \gamma st_{EH} \cdot V_{EH} + \gamma st_{LS} \cdot V_{LS} + \gamma st_{LL} \cdot V_{LL}\right)$$

Exterior Wall	$Vstr1_{XW} = 7.98$	kip
Interior Wall	$Vstr1_{IW} = 0.40$	kip
Top Slab	$Vstr1_{TS} = 12.20$	kip
Bottom Slab	$Vstr1_{BS} = 12.16$	kip

<u>Service 1 Shears</u>

$$V_{s1} = \eta \cdot \left(\gamma s1_{DC} \cdot V_{DC} + \gamma s1_{DW} \cdot V_{DW} + \gamma s1_{EV} \cdot V_{EV} + \gamma s1_{EH} \cdot V_{EH} + \gamma s1_{LS} \cdot V_{LS} + \gamma s1_{LL} \cdot V_{LL}\right)$$

Exterior Wall	$Vs1_{XW} = 5.64$	kip
Interior Wall	$Vs1_{IW} = 0.23$	kip
Top Slab	$Vs1_{TS} = 7.62$	kip
Bottom Slab	$Vs1_{BS} = 7.96$	kip

E36-1.12.2 Concrete Shear Resistance

Check that the nominal shear resistance, V_n, of the concrete in the top slab is adequate for shear without shear reinforcement per **LRFD [5.14.5.3]**.

$$V_n = V_c = \left(0.0676 \cdot \lambda \sqrt{f'_c} + 4.6 \cdot \frac{A_s}{b \cdot d_s} \cdot \frac{V_u \cdot d_s}{M_u}\right) \cdot b \cdot d_s \le 0.126 \cdot \lambda \sqrt{f'_c} \cdot b \cdot d_s$$

$f'_c = 3.5$ culvert concrete strength, ksi

$A_{s\_TS} = 0.15$ area of reinforcing steel in the design width, in^2/ft width

$h := t_{ts}$ height of concrete design section, in $h = 12.50$ in

$\lambda = 1.0$ normal wgt. conc. **LRFD [5.4.2.8]**

Calculate d_s, the distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement:

$$d_s := h - cover - \frac{Bar_D(Bar_{No})}{2} \qquad \boxed{d_s = 10.19} \text{ in}$$

$$V_u := Vstr1_{TS} \qquad \boxed{V_u = 12.2} \text{ kips}$$

$M_u = 264.01$ factored moment occurring simultaneously with V_u, kip-in

$b := 12$ design width, in

For reinforced concrete cast-in-place box structures, $\phi_v = 0.85$, **LRFD [Table 12.5.5-1]**. Therefore the usable capacity is:

$\frac{V_u \cdot d_s}{M_u}$ shall not be taken to be greater than 1.0 $\qquad \frac{V_u \cdot d_s}{M_u} = 0.47 \qquad < 1.0$ OK

$$V_{r1s} := \phi_v \cdot \left[\left(0.0676 \cdot \lambda \sqrt{f'_c} + 4.6 \cdot \frac{A_{s\_TS}}{b \cdot d_s} \cdot \frac{V_u \cdot d_s}{M_u}\right) \cdot b \cdot d_s\right] \qquad \boxed{V_{r1s} = 14.1} \text{ kips}$$

but $\le$ $\quad V_{r2s} := \phi_v \cdot \left(0.126 \cdot \lambda \sqrt{f'_c} \cdot b \cdot d_s\right) \qquad \boxed{V_{r2s} = 24.5}$ kips

$$V_{rs} := \min(V_{r1s}, V_{r2s}) \qquad \boxed{V_{rs} = 14.1} \text{ kips}$$

Check that the provided shear capacity is adequate:

Is $V_u = 12.2$ kip $\le$ $V_{rs} = 14.1$ kip ? $\qquad \boxed{check = \text{"OK"}}$

Note: For single-cell box culverts only, V_c for slabs monolithic with walls need not be taken to be less than: **LRFD[5.14.5.3]** $\qquad 0.0948 \cdot \lambda \sqrt{f'_c} \; b \cdot d_s$

V_c for slabs simply supported need not be taken to be less than: $\qquad 0.0791 \cdot \lambda \sqrt{f'_c} \; b \cdot d_s$

λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

LRFD [5.8] and **LRFD [5.13.3.6]** apply to slabs of box culverts with less than 2.0 ft of fill.

Check that the nominal shear resistance, V_n, of the concrete in the walls is adequate for shear without shear reinforcement per **LRFD [5.8.3.3]**. Calculations shown are for the exterior wall.

$$V_n = V_c = 0.0316 \cdot \beta \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v \le 0.25 \cdot f'_c \cdot b_v \cdot d_v$$

$\beta := 2$ **LRFD [5.8.3.4.1]**

$f'_c = 3.5$ culvert concrete strength, ksi

$b_v := 12$ effective width, in

$h := t_{wex}$ height of concrete design section, in $\qquad h = 12.00$ in

λ = 1.0 normal wgt. conc. **LRFD [5.4.2.8]**

Distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement:

$$d_s := h - cover - \frac{Bar_D(Bar_{No})}{2} \qquad \boxed{d_s = 9.69} \quad in$$

The effective shear depth taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure; **LRFD [5.8.2.9]**

$$d_{v\_i} = d_s - \frac{a}{2}$$

from earlier calculations:

$\boxed{\beta_1 = 0.85}$

$\boxed{f_s = 60}$ ksi

$\boxed{A_{s\_XW} = 0.40}$ in^2

The distance between the neutral axis and the compression face:

$$c := \frac{A_{s\_XW} \cdot f_s}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b_v} \qquad \boxed{\beta_1 = 0.85} \; \boxed{\alpha_1 = 0.85} \qquad \boxed{c = 0.79} \quad in$$

$$a := \beta_1 \cdot c \qquad \boxed{a = 0.67} \quad in$$

The effective shear depth:

$$d_{v\_i} := \left(d_s - \frac{a}{2}\right) \qquad \boxed{d_{v\_i} = 9.35}$$

d_v need not be taken to be less than the greater of 0.9 d_s or 0.72h (in.)

$$d_v := \max\left(d_{v\_i}, \max\left(0.9 d_s, 0.72 t_{wex}\right)\right) \qquad 0.9 \cdot d_s = 8.72$$

$$d_v = 9.35 \quad in \qquad 0.72 \cdot t_{wex} = 8.64$$

For reinforced concrete cast-in-place box structures, $\phi_v = 0.85$, **LRFD [Table 12.5.5-1]**.

Therefore the usable capacity is:

λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$$V_{r1w} := \phi_v \cdot \left(0.0316 \cdot \beta \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v\right) \qquad \boxed{V_{r1w} = 11} \quad kips$$

$$\text{but} \le \quad V_{r2w} := \phi_v \cdot \left(0.25 \cdot f'_c \cdot b_v \cdot d_v\right) \qquad \boxed{V_{r2w} = 83} \quad kips$$

$$V_{rw} := \min\left(V_{r1w}, V_{r2w}\right) \qquad \boxed{V_{rw} = 11} \quad kips$$

$$V_u := Vstr1_{XW} \qquad \boxed{V_u = 8.0} \quad kips$$

Check that the provided shear capacity is adequate:

Is $V_u = 8.0$ kip $\le$ $V_{rw} = 11.3$ kip ? $\boxed{check = \text{"OK"}}$

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37.1 Structure Selection

Most pedestrian bridges are located in urban areas and carry pedestrian and/or bicycle traffic over divided highways, expressways and freeway systems. The structure type selected is made on the basis of aesthetics and economic considerations. A wide variety of structure types are available and each type is defined by the superstructure used. Some of the more common types are as follows:

- Concrete Slab
- Prestressed Concrete Girder
- Steel Girder
- Prefabricated Truss

Several pedestrian bridges are a combination of two structure types such as a concrete slab approach span and steel girder center spans. One of the more unique pedestrian structures in Wisconsin is a cable stayed bridge. This structure was built in 1970 over USH 41 in Menomonee Falls. It is the first known cable stayed bridge constructed in the United States. Generally, pedestrian bridges provide the designer the opportunity to employ long spans and medium depth sections to achieve a graceful structure.

Pedestrian boardwalks will not be considered “bridges” when their clear spans are less than or equal to 20 feet, and their height above ground and/or water is less than 10 feet. Boardwalks falling under these constraints will not be required to follow the design requirements in the WisDOT Bridge Manual, but will need to follow the standards established in the *Wisconsin Bicycle Facility Design Handbook.*

37.2 Specifications and Standards

The designer shall refer to the following related specifications:

- "*AASHTO LRFD Bridge Design Specifications*"
- *"AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges",* hereafter referred to as the *"Pedestrian Bridge Guide"*
- See *Standardized Special Provision (STSP)* titled "*Prefabricated Steel Truss Pedestrian Bridge LRFD*" for the requirements for this bridge type

For additional design information, refer to the appropriate Wisconsin Bridge Manual chapters relative to the structure type selected.

The pedestrian live load (PL) shall be as follows: (from "*Pedestrian Bridge Guide")*

- 90 psf **[Article 3.1]**
- Dynamic load allowance is not applied to pedestrian live loads **[Article 3.1]**

The vehicle live load shall be applied as follows: (from *"Pedestrian Bridge Guide")*

- Design for an occasional single maintenance vehicle live load (LL) **[Article 3.2]**

Clear Bridge Width (w)	Maintenance Vehicle
7 ft $\leq$ w $\leq$ 10 ft	H5 Truck (10,000 lbs)
w > 10 ft	H10 Truck (20,000 lbs)

- Clear bridge widths of less than 7 feet need not be designed for maintenance vehicles. **[Article 3.2]**
- The maintenance vehicle live load shall not be placed in combination with the pedestrian live load. **[Article 3.2]**
- Dynamic load allowance is not applied to the maintenance vehicle. **[Article 3.2]**
- Strength I Limit State shall be used for the maintenance vehicle loading. **[Article 3.2, 3.7]**

On Federal Aid Structures FHWA requests a limiting gradient of 8.33 percent (1:12) on ramps for pedestrian facilities to accommodate the physically handicapped and elderly as recommended by the "*American Standard Specifications for Making Buildings and Other Facilities Accessible to, and Usable by, the Physically Handicapped*". This is slightly flatter than the gradient guidelines set by AASHTO which states gradients on ramps should not be more than 15 percent and preferably not steeper than 10 percent.

The minimum inside clear width of a pedestrian bridge on a pedestrian accessible route is 8 feet. (*AASHTO Guide for the Planning, Design, and Operation of Pedestrian Facilities, 2004).*

The width required is based on the type, volume, and direction of pedestrian and/or bicycle traffic.

The vertical clearance on the pedestrian bridge shall be a minimum of 10 feet for bicyclists' comfort and to allow access for maintenance and emergency vehicles. The Wisconsin Department of Natural Resources recommends a vertical clearance on the bridge of at least 12 feet to accommodate maintenance and snow grooming equipment on state trails. Before beginning the design of the structure, the Department of Natural Resources and the Bureau of Structures should be contacted for the vertical clearance requirements for all vehicles that require access to the bridge.

In addition, ramps should have rest areas or landings 5 feet to 6 feet in length which are level and safe. Rest area landings are mandatory when the ramp gradient exceeds 5 percent. Recommendations are that landings be spaced at 30 foot maximum intervals, as well as wherever a ramp turns. This value is based on a maximum gradient of 8.33 percent on pedestrian ramps, and limiting ramps to a maximum rise of 30 inches per ramp run. Also, ramps are required to have handrails on both sides. See Standard Details for handrail location and details.

Minimum vertical clearance for a pedestrian overpass can be found in the *Facilities Development Manual (FDM)* Procedure 11-35-1, Attachment 1.8 and 1.9. Horizontal clearance is provided in accordance with the requirement found in *(FDM)* Procedure 11-35-1, Attachment 1.5 and 1.6.

Live load deflection limits shall be in accordance with the provisions of **LRFD [2.5.2.6.2]** for the appropriate structure type.

Pedestrian loads, as described in the AASHTO LRFD Bridge Design Specifications, shall be used to not only design the pedestrian railings on the structure, but shall also be used to design stairway railings that are adjacent to the structure and are part of the contract.

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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

<u>Table 40.16-1</u>
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. <u>For mechanical anchors:</u>

$$N_r = \phi_{ts}N_{sa} \le \phi_{tc}N_{cb} \le \phi_{tc}N_{pn}$$

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)

≤ $1.9f_{ya}$

≤ 125 ksi

f_{ya} = Specified yield strength of anchor steel (psi)

ϕ_{tc} = Strength reduction factor for anchors in concrete

= 0.65 for anchors without supplementary reinforcement per 40.16.2

= 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 40.1

$$= (S_1 + S_2)(S_3 + S_4)$$

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 40.1 (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 $c_{a,min} \geq c_{ac}$

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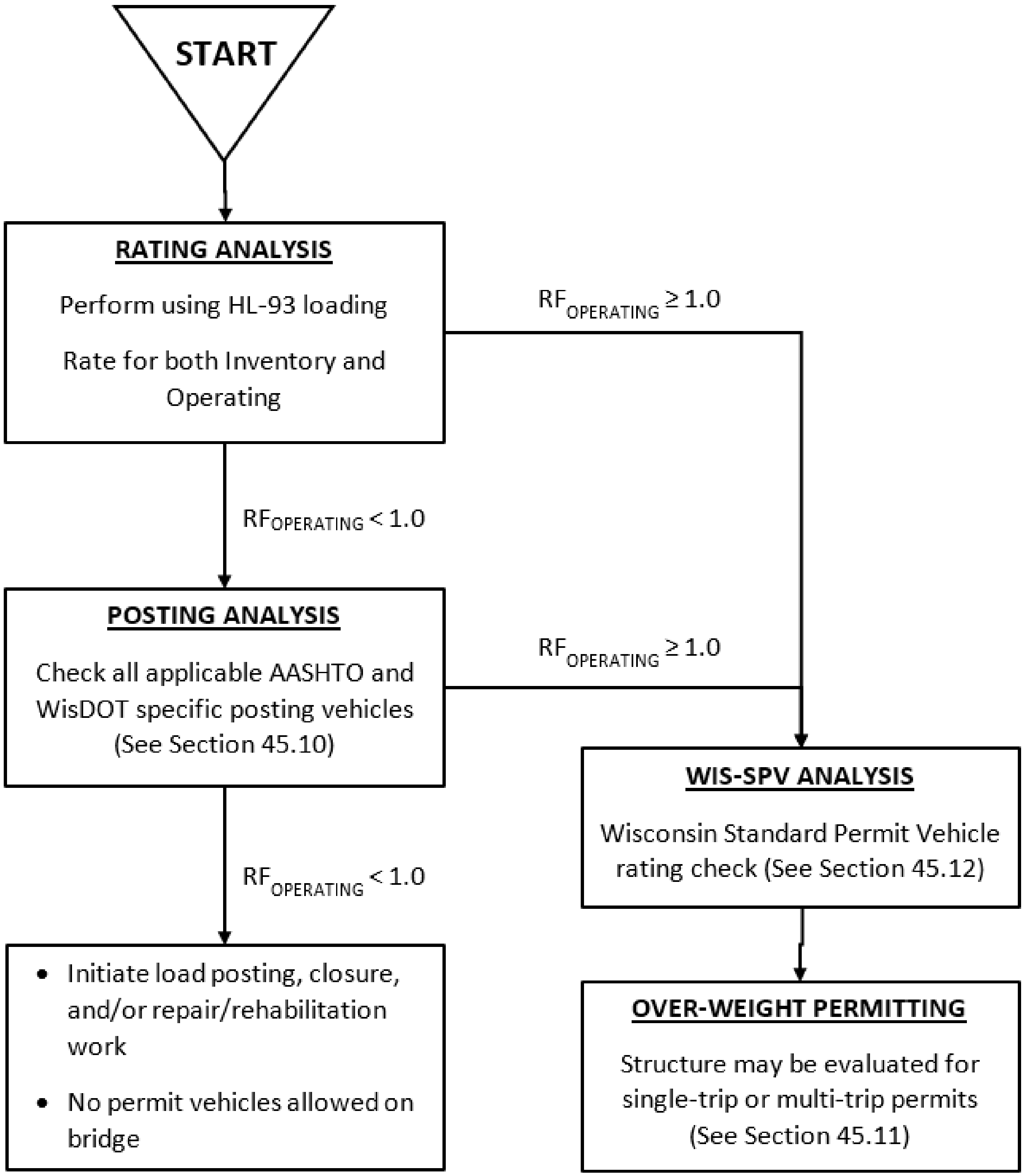


Figure 45.3-1
Load and Resistance Factor Rating Flow Chart

45.3.7.2 Load Factors

The load factors for the Design Load Rating shall be taken as shown in Table 45.3-1. The load factors for the Legal Load Rating shall be taken as shown in Table 45.3-1 and Table 45.3-2. The load factors for the Permit Load Rating shall be taken as shown in Table 45.3-1 and Table 45.3-3 . Again, note that the shaded values in Table 45.3-1 indicate optional checks that are currently not required by WisDOT.

Bridge Type	Limit State	Dead Load DC	Dead Load DW	Design Load		Legal Load	Permit Load
				Inventory	Operating		
				LL	LL	LL	LL
Steel	Strength I, II	1.25	1.50	1.75	1.35	Table 45.3-2	Table 45.3-3
	Service II	1.00	1.00	1.30	1.00	1.30	1.00
Reinforced Concrete	Strength I, II	1.25	1.50	1.75	1.35	Table 45.3-2	Table 45.3-3
	Service I	1.00	1.00	--	--	--	1.00
Prestressed Concrete	Strength I, II	1.25	1.50	1.75	1.35	Table 45.3-2	Table 45.3-3
	Service III	1.00	1.00	0.80	--	1.00	--
	Service I	1.00	1.00	--	--	--	1.00
Timber	Strength I, II	1.25	1.50	1.75	1.35	Table 45.3-2	Table 45.3-3

Table 45.3-1
Limit States and Live Load Factors (γ_{LL}) for LRFR

Loading Type	Live Load Factor
AASHTO Legal Vehicles, State Specific Vehicles, and Lane Type Legal Load Models	1.45
Specialized Haul Vehicles (SU4, SU5, SU6, SU7)	1.45

Table 45.3-2
Live Load Factors (γ_{LL}) for Legal Loads in LRFR

45.3.10 Engineering Judgment, Condition-Based Ratings, and Load Testing

Engineering judgment or condition-based ratings alone shall not be used to determine the capacity of a bridge when sufficient structural information is available to perform a calculation-based method of analysis.

Ratings determined by the method of field evaluation and documented engineering judgment may be considered when the capacity cannot be calculated due to one or more of the following reasons:

- No bridge plans available
- Concrete bridges with unknown reinforcement

The engineer shall consider all available information, including:

- Condition of load carrying elements (inspection reports – current and historic)
- Year of construction
- Material properties of members (known or assumed per 45.5.2)
- Type of construction
- Redundancy of load path
- Field measurements
- Comparable structures with known construction details
- Changes since original construction
- Loading (past, present, and future)
- Other information that may contribute to making a more-informed decision

If the engineer of record is considering using a judgment- or inspection-based load rating, a thorough visual observation of the bridge should be conducted, including observing actual traffic patterns for the in-service bridge.

The criteria applied to determine a rating by field evaluation and documented engineering judgment shall be documented via the Load Rating Summary Form (see 45.9) accompanied by any and all related inspection reports, any calculation performed to assist in the rating and assumptions used for those calculations, a written description of the observed traffic patterns for the bridge, relevant correspondences, and any available, relevant photographs of the bridge or bridge condition.

Bridge owners may also consider nondestructive proof load tests in order to establish a safe capacity for bridges in which a load rating cannot be calculated.

WisDOT policy items:

Consult the Bureau of Structures Rating Unit before moving forward with an engineering judgment-based, inspection-based load rating, or with a load testing procedure on either the State or Local system.

45.3.11 Refined Analysis

Methods of refined analysis are discussed in **LRFD [4.6.3]**. These include the use of 2D and 3D finite element modeling of bridge structures, which preclude the use of live load distribution factor equations and instead rely on the relative stiffness of elements in the analytical model for distribution of applied loads. As such, a 2D or 3D model requires the inclusion of elements contributing to the transverse distribution of loads, such as deck and cross frame elements that are otherwise not directly considered in a line girder or strip width analysis. Additional guidance on refined analysis can be found in the AASHTO/NSBA publication "G13.1 Guidelines for Steel Girder Bridge Analysis, 2nd Edition" and the FHWA "Manual of Refined Analysis" (anticipated 2017).

WisDOT policy items:

Prior to using refined analysis, consult the Bureau of Structures Rating Unit. Additional documentation is required when performing a refined analysis; see 45.9 for these requirements.

The Bureau of Structures does not require a specific piece of software be used by consultant engineers when performing a refined load rating analysis. See 45.4 for information on load rating computer software.

Refined analysis for load rating purposes is required for certain structure types, and/or structures with certain geometric characteristics. In other instances a refined analysis may be utilized to improve the structure rating for the purpose of avoiding load posting or to improve the capacity for permitting.

A refined analysis is required for the following structure types:

- Steel rigid frames
- Bascule-type movable bridges
- Tied arches
- Cable stayed or suspension bridges
- Steel box (tub) girder bridges

A refined analysis is require if any of the following geometric characteristics are present within the structural system to be load rated:

- Steel girder structure curved in plan, not meeting the criteria discussed in 45.6.3.2.1.

- Steel girder structure skewed 40 degrees or more, with cross framing type discussed in 45.6.3.2.2.

- Skew varies between adjacent supports by more than 20 degrees.

A refined analysis *may* be required if any of the following geometric characteristics are present within the structural system to be load rated. Contact the Bureau of Structures Rating Unit prior to determine the level of effort to rate the structure.

- Steel girder structures with flared girder spacing, such that the change in girder spacing over the span length is greater or equal to 0.015 ($\Delta S/L \geq 0.015$).

- Structures with complex framing plans; those having discontinuous girders utilizing transfer girders in-span.

- Superstructure supported by flexible supports (e.g. straddle bent with integral pier cap). *Note: These "flexible" supports are considered primary members and are to be included in a load rating.*

45.4 Load Rating Computer Software

Though not required, computer software is a common tool used for load rating. WisDOT BOS encourages the use of software for its benefits in increased efficiency and accuracy. However, the load rating engineer must be aware that software is a tool; the engineer maintains responsibility for understanding and verifying any load rating obtained from computer software and should have a full understanding of all underlying assumptions. The load rating engineer is responsible for ensuring that any software used to develop a rating performs that rating in accordance with relevant AASHTO specifications and taking into account specific WisDOT policy noted in this chapter.

45.4.1 Rating Software Utilized by WisDOT

The Bureau of Structures currently uses a mix of software developed in-house and software available commercially. For a list of software currently used by WisDOT for each primary structure type, see the Bureau of Structures website:

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

WisDOT does not currently mandate the use of any particular software for load ratings.

45.4.2 Computer Software File Submittal Requirements

When load rating software is used as a tool to derive the load rating for a bridge project (new or rehabilitation), the electronic input file shall be included with the project submittal.

Some superstructure types may require advanced modeling techniques in order to fully and accurately capture the structural response. See 45.3.11 for more information on refined analysis.

See 45.9 (Documentation and Submittals) for more information.

45.5 General Requirements

45.5.1 Loads

45.5.1.1 Material Unit Weights

The following assumptions for material unit weights shall be used when performing a load rating, unless there is project-specific information.

Asphalt	145 pcf
Reinforced Concrete	150 pcf
Soil or Gravel	120 pcf
Steel	490 pcf
Water	62.4 pcf
Timber	50 pcf
½" Thin Epoxy Overlay	5 psf

45.5.1.2 Live Loads

Live loads shall be per 45.3.7 (LRFR), 45.3.8 (LFR), and 45.3.9 (ASR).

WisDOT policy items:

Inventory and operating ratings shall consider the possibility of truck loads on sidewalks. However, posting and permitting analysis need not be calculated with wheel placement on sidewalks.

Lane placement in accordance with AASHTO design specifications may not be consistent with actual usage of a bridge as defined by its striped lanes, and could result in conservative load ratings for bridge types such as trusses, two-girder bridges, ramp structures, arches and bridges with exterior girders governing the ratings via lever rule live load distribution assumptions.

WisDOT policy items:

Upon the approval of the Bureau of Structures Rating Unit, a load rating may be performed by placing truck loads only within the striped lanes. When this alternative is utilized, placement of striped lanes on the bridge shall be field verified and documented in the inspection report, per **MBE [6A.2.3.2]** and **[6B.6.2.2]**.

45.5.1.3 Dead Loads

Dead loads are determined based on the weight and dimensions of the elements in question and shall be distributed as noted in sections above. The following is further guidance offered by WisDOT related to various dead loads.

- The top ½" (or greater if a concrete overlay was placed integral with the deck at the time of pour) of a monolithic concrete deck should be considered a wearing surface. It shall not be considered structural, and thus not used to compute section properties or for composite action.

- For an overlay placed integral with the deck at the time of original construction, the overlay thickness shall be considered a wearing surface. It should not be considered structural, and thus not used to compute section properties or for composite action.

- For a bridge with an existing overlay, only the full remaining thickness of the original deck (original thickness – thickness milled off during overlay process) may be considered structural.

- If the design of a new bridge includes an allowance for a future wearing surface, parapets, sidewalks, or other dead loads, that load shall be excluded during the load rating. A load rating is considered a snapshot of current capacity and should only include loads actually in-place at the time of the rating.

- The weight of the concrete haunch for girder superstructures should be included in the non-composite dead load. The actual average haunch height may be used for load calculations. It is also acceptable to calculate the haunch dead load effect assuming the haunch thickness to vary along the length of the beam, if actual, precise haunch thicknesses are known.

45.5.2 Material Structural Properties

Material properties shall be as stated in AASHTO *MBE* or as stated in this chapter. Often when rating a structure without a complete set of plans, material properties are unknown. The following section can be used as a guideline for the rating engineer when dealing with structures with unknown material properties. If necessary, material testing may be needed to analyze a structure.

45.5.2.1 Reinforcing Steel

The allowable unit stresses and yield strengths for reinforcing steel can be found in Table 45.5-1. When the condition of the steel is unknown, they may be used without reduction. Note that Wisconsin started to use Grade 40 bar steel about 1955-1958; this should be noted on the plans.

Reinforcing Steel Grade	Inventory Allowable (psi)	Operating Allowable (psi)	Minimum Yield Point (psi)
Unknown	18,000	25,000	33,000
Structural Grade	19,800	27,000	36,000
Grade 40 (Intermediate)	20,000	28,000	40,000
Grade 60	24,000	36,000	60,000

Table 45.5-1
Yield Strength of Reinforcing Steel

45.5.2.2 Concrete

The following are the maximum allowable unit stresses in concrete in pounds per square inch (see Table 45.5-2). Note that the "Year Built" column may be used if concrete strength is not available from the structure plans.

Year Built	Inventory Allowable (psi)	Operating Allowable (psi)	Compressive Strength (F'c) (psi)	Modular Ratio
Before 1959	1000	1500	2500	12
1959 and later	1400	1900	3500	10
For all non-prestressed slabs 1975 and later	1600	2400	4000	8
Prestressed girders before 1964 and all prestressed slabs	2000	3000	5000	6
1964 and later for prestressed girders	2400	3000	6000	5

Table 45.5-2
Minimum Compressive Strengths of Concrete

45.5.2.3 Prestressing Steel Strands

Table 45.5-3 contains values for uncoated Seven-Wire Stressed-Relieved and Low Relaxation Strands:

Year Built	Grade	Nominal Diameter of Strand (In)	Nominal Steel Area of Strand (In^2)	Yield Strength (psi)	Breaking Strength (psi)
Prior To 1963	250	$^7/_{16}$ (0.438)	0.108	213,000	250,000
Prior To 1963	250	½ (0.500)	0.144	212,500	250,000
1963 To Present	270	½ (0.500)	0.153	229,000	270,000
1973 To Present	270 Low Relaxation	½ (0.500)	0.153	242,500	270,000
1995 to Present	270 Low Relaxation	$^9/_{16}$ (0.600)	0.217	242,500	270,000

Table 45.5-3
Tensile Strength of Prestressing Strands

The "Year Built" column is for informational purposes only. The actual diameter of strand and grade should be obtained from the structure plans.

45.5.2.4 Structural Steel

The **MBE [Table 6B.5.2.1-1]** gives allowable stresses for steel based on year of construction or known type of steel. For newer bridges, refer to AASHTO design specifications.

<table>
<tr><th colspan="2">Steel Type</th><th>AASHTO Designation</th><th>ASTM Designation</th><th>Minimum Tensile Strength, Fu (psi)</th><th>Minimum Yield Strength, Fy (psi)</th></tr>
<tr><td rowspan="4">Unknown Steel</td><td>Built prior to 1905</td><td></td><td></td><td>52,000</td><td>26,000</td></tr>
<tr><td>1905 to 1936</td><td></td><td></td><td>60,000</td><td>30,000</td></tr>
<tr><td>1936 to 1963</td><td></td><td></td><td></td><td>33,000</td></tr>
<tr><td>After 1963</td><td></td><td></td><td></td><td>36,000</td></tr>
<tr><td colspan="2">Carbon Steel</td><td>M 94 (1961)</td><td>A 7 (1967)</td><td>60,000</td><td>33,000</td></tr>
<tr><td colspan="2">Nickel Steel</td><td>M 96 (1961)</td><td>A 8 (1961)</td><td>90,000</td><td>55,000</td></tr>
<tr><td rowspan="3">Silicon Steel</td><td>up to 1-1/8" thick</td><td>M 95 (1961)</td><td>A 94</td><td>75,000</td><td>50,000</td></tr>
<tr><td>1-1/8" to 2" thick</td><td></td><td>A 94</td><td>72,000</td><td>47,000</td></tr>
<tr><td>2" to 4" thick</td><td></td><td>A 94 (1966)</td><td>70,000</td><td>45,000</td></tr>
<tr><td>Low Alloy Steel</td><td></td><td></td><td>A441</td><td>75,000</td><td>50,000</td></tr>
</table>

Table 45.5-4
Minimum Yield Strength Values for Common Steel Types

45.5.2.5 Timber

If plans are available, values and adjustment factors will be taken from the most recent edition of the *National Design Specifications for Wood Construction* (NDS) based on the species and grade of the timber as given on the plans. On older plans that may give the stresses, the stress used for the ratings will be the values from the NDS that correspond with the applicable capacity provisions. If plans are not available, Table 45.5-5 shall be used to estimate the allowable stresses.

For operating ratings, all stresses, in determining capacity, will be multiplied by 1.33.

Bridge Type	Component	Species and Grade	Bending Stress (F_b), psi	Shear Stress (F_v), psi
Longitudinal Nail Laminated Slab Bridges	Slab	Douglas Fir-Larch No. 1 & Btr NDS 2012 Table 4A	1200	180
Longitudinal Glued Laminated Slab Bridges	Slab	20F-V7 NDS 2012 Table 5A	2000	265
Girder-Deck Bridges	Girder, Glu-lam	20F-V7 NDS 2012 Table 5A	2000	265
	Girder, Solid-Sawn	Douglas Fir-Larch Select Structural NDS 2012 Table 4D	1600	170
	Transverse Deck, Glulam	20F-V7 NDS 2012 Table 5A	1600	265
	Transverse Deck, Solid-Sawn	Douglas Fir-Larch No. 1 & Btr NDS 2012 Table 4A	1200	180
Longitudinal Stress-laminated Bridges	Slab, Glu-lam	20F-V7 NDS 2012 Table 5A	2000	265
	Slab, Solid Sawn	Douglas Fir-Larch No. 1 & Btr NDS 2012 Table 4A	1200	180
Substructure Components		**Species and Grade**	**Compression Stress (F_C) psi**	**E_{min} psi**
Timber Piles		Pacific Coast Douglas Fir NDS 2012 Table 6A	1300	690,000

Table 45.5-5
Maximum Allowable Stress for Timber Components

45.5.2.5.1 Timber Adjustment Factors

The following is guidance offered by WisDOT related to timber adjustment factors.

- Load Duration (C_D): Bending, shear, and compression stresses shall be multiplied by 1.15 (traffic load duration).
- Wet Service (C_M): Bending and shear stresses shall be multiplied by the appropriate factor per the footnotes in NDS by assuming that the timber is wet in service. An exception to this is if the rating engineer considers the deck's surface to be impervious,

then C_M shall be 1.0. For large glulam girders covered with deck and wearing surface in good condition such that the girders remain dry, C_M = 1.0.

- Beam Stability (C_L): All girders with decks fastened in the normal manner shall be assumed to have continuous lateral stability and C_L shall be 1.0. If the girders are not prevented from rotating at the points of bearing, or rating engineer determines that there is not continuous lateral support on the compression edge, C_L shall be determined by **NDS [3.3.3]**.

- Size (C_F): Bending stresses for sawn lumber shall be multiplied by the appropriate factor per the footnotes in NDS.

- Volume (C_v): Bending stresses for glued laminated timber shall be multiplied by the appropriate factor per the footnotes in NDS.

- Flat Use (C_{fu}): Bending stresses shall be multiplied by the appropriate factor per NDS, for plank decking loaded on the wide face.

- Repetitive Member (C_r): Bending stresses shall be multiplied by 1.15 on longitudinal nail laminated bridges and on nail laminated decks. For deck planks, 1.15 may be used if they are covered by bituminous surface or perpendicular planks for load distribution and are spaced not more than 24" on center.

- Condition Treatment Factor (C_{pt}): Piling, Bending, Shear, and Compression stresses shall be multiplied by: 1.0 for all douglas fir pile installed prior to 1985, and by 0.9 for all other piles.

- Load Sharing Factor (C_{ls}): This shall be typically be 1.0 for all bents. A higher value may be used per **NDS [6.3.11]** when multiple piles are connected by concrete caps or equivalent force distributing elements so that the pile group deforms together.

- Column Stability (C_P): Compression stresses in bents shall by multiplied by C_p per **NDS [3.7]**. "d" in the formula shall be the minimum measured remaining pile dimension. Unless determined otherwise by the rating engineer, it shall be assumed that all the piles shall have the area and C_p of the worst pile.

The adjusted allowable stress used in ratings shall be the given stress multiplied by all the applicable adjustment factors.

45.6 WisDOT Load Rating Policy and Procedure – Superstructure

This section contains WisDOT policy items or guidance related to the load rating of various types of bridge superstructures.

45.6.1 Prestressed Concrete

For bridges designed to be continuous over interior supports, the negative capacity shall come from the reinforcing steel in the concrete deck. Conservatively, only the top mat of steel deck reinforcing steel should be considered when rating for negative moment. If this assumption results in abnormally low ratings for negative moment, contact the Bureau of Structures Rating Unit for consultation.

Elastic gains in prestressed concrete elements shall be neglected for a conservative approach.

Shear design equations for prestressed concrete bridges have evolved through various revisions of the AASHTO design code. Because of this, prestressed concrete bridges designed during the 1960s and 1970s may not meet current shear capacity requirements. Shear capacity should be calculated based on the most current AASHTO code, either LFR or LRFR. Shear should be considered when determining the controlling ratings for a structure. If shear capacities are determined to be insufficient, the load rating engineer of record should contact the Bureau of Structures Rating Unit for consultation.

If an option is given on the structure plans to use either stress relieved or low relaxation strand, or $^{7}/_{16}$" or $^{1}/_{2}$" diameter strand, consult the shop drawings for the structure to see which option was exercised. If the shop drawings are not available, all possible options should be analyzed and the option which gives the lowest operating rating should be reported.

45.6.1.1 I-Girder

Bridges may have varying girder spacing between spans. A common configuration in Wisconsin with prestressed I-girder superstructures is a four-span bridge with continuous girders in spans 2 & 3 and different (wider) girder spacing in spans 1 & 4. If the chosen analysis program is unable to handle girder spacing varying between spans, several analysis runs may need to be done to capture all potential controlling scenarios.

- In the scenario described above, it seems to have been common practice in the past that when the structure received a new deck, the deck would be poured continuous over all four spans, with negative moment reinforcing in the deck included over the piers. If a full-depth concrete diaphragm is present at the piers, it is acceptable practice to rate the structure as a four-span continuous structure. It is also acceptable to rate the structure as originally constructed; simple exterior spans and two interior spans that are continuous. The decision on how to consider this structure configuration is at the discretion of the rating engineer. All assumptions made should be clearly noted in the calculations and in the load rating summary sheet (See Section 45.9.1).

When the shear failure plane crosses multiple stirrup zones, guidance given in the **MBE [6A.5.8]** should be followed to determine an average shear reinforcement area per unit length existing within the shear failure plane. The shear failure plane is assumed to cross through mid-depth of the section with a 45 degree angle of inclination.

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

45.6.1.1.1 Variable Girder Spacing (Flare)

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

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

45.6.1.2 Box and Channel Girders

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

45.6.2 Cast-in-Place Concrete

45.6.2.1 Slab (Flat or Haunched)

When using Load Factor Rating (LFR) or Allowable Stress Rating (ASR) and calculating the single lane load distribution factor for concrete slab bridges, the distribution width, E, shall be taken as 12'-0".

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

45.6.3 Steel

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

Plastic analysis shall be used for steel members as permitted by AASHTO specifications, including (but not limited to) Article 6.12.2 (LRFR) and Articles 10.48.1, 10.53.1.1, and 10.54.2.1 (LFR). Plastic analysis shall not be used for members with significant deterioration. Per code, sections must be properly braced in order to consider plastic capacity. For questions on the use of plastic analysis, contact the Bureau of Structures Rating Unit.

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

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

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

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

WisDOT policy items:

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

45.6.3.1 Fatigue

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

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

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

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

W_e/S, where:

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

S = Average stringer spacing in feet.

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

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

45.6.3.2.1 Curvature and/or Kinked Girders

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

45.6.3.2.2 Skew

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

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

45.6.3.2.3 Variable Girder Spacing (Flare)

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

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

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

45.6.3.3 Truss

45.6.3.3.1 Gusset Plates

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

45.6.3.4 Bascule-Type Movable Bridges

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

45.6.4 Timber

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

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

45.6.4.1 Timber Slab

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

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

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

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

45.7 WisDOT Load Rating Policy and Procedure – Substructure

45.7.1 Timber Pile Abutments and Bents

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

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

45.8 WisDOT Load Rating Policy and Procedure – Culverts

45.8.1 Rating New Culverts

Ratings for new bridge-length (assigned a B number) culverts should be determined based on culvert type. See below for more guidance and see 45.9 for documentation and submittal requirements.

The Bureau of Structures does not currently require rating calculations to be submitted for new culverts that are not of bridge-length. However, these may be required in the future. When they are designed with software that readily produces load ratings, those ratings should be included on plan and calculation submittals. As a minimum, the design vehicle and design overburden depth shall be shown on plans. When load ratings are not calculated, ratings shall be reported as:

- Inventory rating factor: 1.05
- Operating rating factor: 1.35
- Wisconsin Standard Permit Vehicle (Wis-SPV): 255 kips

45.8.1.1 New Concrete Box Culverts

Concrete box culverts shall be load rated per AASHTO specifications.

The fill depth in relation to the structure dimensions will determine the live load effect on the structure. For structures that experience little or no live load based on analysis, the ratings reported on plans and in the load rating summary form shall not exceed the ratings noted below:

- Inventory rating factor: 2.0
- Operating rating factor: 3.0
- Wisconsin Standard Permit Vehicle (Wis-SPV): 250 kips

45.8.1.2 New Concrete Pipe Culverts

A concrete pipe culvert system (culvert and fill) shall be designed to carry HL-93 loading and the Wis-SPV as described in 36.1.3. Ratings shall be reported as:

- Inventory rating factor: 1.0
- Operating rating factor: 1.67
- Wisconsin Standard Permit Vehicle (Wis-SPV): 190 kips

45.8.1.3 New Steel Pipe Culverts

A steel pipe culvert system (culvert and fill) shall be designed to carry HL-93 loading and the Wis-SPV as described in 36.1.3. Ratings shall be reported as:

- Inventory rating factor: 1.0
- Operating rating factor: 1.67
- Wisconsin Standard Permit Vehicle (Wis-SPV): 190 kips

45.8.2 Rating Existing (In-Service) Culverts

Ratings for existing (in-service) bridge-length culverts shall be determined based on culvert type and the depth of fill over the culvert. See below for more guidance and see 45.9 for documentation and submittal requirements.

The Bureau of Structures currently does not require rating calculations to be submitted for in-service culverts that are not of bridge-length. If deterioration or other significant changes warrant consideration of a load posting for an in-service culvert that is not of bridge-length, contact the Bureau of Structures on what is required for a load posting evaluation.

45.8.2.1 In-Service Concrete Box Culverts

In-service concrete box culverts with 6'-0" or less of fill may require a load rating. In-service concrete box culverts with more than 6'-0" of fill over the top slab and in fair or better condition based on the most recent inspection report do not require a calculated load rating. Ratings shall be reported as:

- Inventory rating factor: 1.0
- Operating rating factor: 1.67
- Wisconsin Standard Permit Vehicle (Wis-SPV): 190 kips

WisDOT policy items:

For in-service concrete boxes with less than 6'-0" of fill or with more than 6'-0" of fill, but in poor condition, contact the Bureau of Structures Rating Unit for direction on what is required for a load rating.

45.8.2.2 In-Service Concrete Pipe Culverts

An in-service concrete pipe culvert in fair or better condition does not require a calculated load rating. Ratings shall be reported as:

- Inventory rating factor: 1.0
- Operating rating factor: 1.67
- Wisconsin Standard Permit Vehicle (Wis-SPV): 190 kips

WisDOT policy items:

For in-service concrete pipe culverts in poor condition, contact the Bureau of Structures Rating Unit for direction on what is required for a load rating.

45.8.2.3 In-Service Steel Pipe Culverts

An in-service steel pipe culvert in fair or better condition does not require a calculated load rating. Ratings shall be reported as:

- Inventory rating factor: 1.0
- Operating rating factor: 1.67
- Wisconsin Standard Permit Vehicle (Wis-SPV): 190 kips

WisDOT policy items:

For in-service steel pipe culverts in poor condition, contact the Bureau of Structures Rating Unit for direction on what is required for a load rating.

45.9 Load Rating Documentation and Submittals

The Bridge Rating and Management Unit is responsible for maintaining information for every structure in the Wisconsin inventory, including load ratings. This information is used throughout the life of the structure to help inform decisions on potential load postings, repairs, rehabilitation, and eventual structure replacement. That being the case, it is critical that WisDOT collect and store complete and accurate documentation regarding load ratings.

45.9.1 Load Rating Calculations

The rating engineer is required to submit load rating calculations. Calculations should be comprehensive and presented in a logical, organized manner. The submitted calculations should include a summary of all assumptions used (if any) to derive the load rating.

45.9.2 Load Rating Summary Forms

After the structure has been load rated, the WisDOT Bridge Load Rating Summary Form shall be completed and utilized as a cover sheet for the load rating calculations (see Figure 45.9-1). This form may be obtained from the Bureau of Structures or is available on the following website:

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

If required, the Refined Analysis Rating Form (see 45.9.5 and Figure 45.9-2) is available at the same location.

If required, the Culvert Load Rating Summary Form (Figure 45.9-3) is available at the same location.

Instructions for completing the forms are as follows:

Load Rating Summary Form

1. Fill out applicable Bridge Data, Structure Type, and Construction History information using HSIS as reference.

2. Check what rating method and rating vehicle was used to rate the bridge in the spaces provided.

3. Enter the inventory/operating ratings, controlling element, controlling force effect, and live load distribution factor for the rating vehicle.

    a. If the load distribution was determined through refined methods (i.e., finite element analysis), it is not necessary to record the live load distribution factor. Instead, enter “REFINED” in the space provided and use the “Remarks/Recommendations” section to describe the methods used to determine live load distribution.

4. The rating for the Wisconsin Special Permit vehicle (Wis-SPV) is always required and shall be given on the rating sheet for both a multi-lane distribution and a single-lane distribution. Make sure not to include the future wearing surface in these calculations. All reported ratings are based on current conditions and do not reflect future wearing surfaces. Enter the Maximum Vehicle Weight (MVW) for the Wis-SPV analysis, controlling element, controlling force effect, and live load distribution factor.

5. When necessary, posting vehicles shall be analyzed and load postings determined per the requirements of 45.10.

   a. Enter the lowest operating rating in kips for each appropriate vehicle type, along with corresponding controlling element and force effect, as well as live load distribution factor.

6. If a posting vehicle analysis was performed, check the box indicating if a load posting is required or not required. If analysis shows that a load posting is required, specify the level of posting and contact the Bureau of Structures Rating Unit immediately.

7. Enter all additional remarks as required to clarify the load capacity calculations.

8. It is necessary for the responsible engineer to sign and seal the form in the space provided. This is true even for rehabilitation projects with no change to the ratings.

Culvert Load Rating Summary Form

1. Engineered, cast-in-place box culverts should use the Load Rating Summary Form. The Culvert Load Rating Summary Form is intended for other culvert types, including pipe culverts, arch culverts, and precast concrete box culverts.

2. Design overburden depth should be taken from the design calculations/documents.

3. Overburden depth is the current, in-service depth of overburden on the culvert structure.

4. If load ratings are available, they should be recorded. If load ratings are unknown, see 45.8 for direction.

45.9.3 Load Rating on Plans

The plans shall contain the following rating information:

- Inventory Load Rating – The plans shall have either the HS value of the inventory rating if using LFR or the rating factor for the HL-93 if using LRFR. For LFR ratings, the rating should be rounded down to the nearest whole number. This rating shall be based on the current conditions of the bridge at the point when the construction is complete and shall not use the future wearing surface. See 6.2.2.3.4 for more information on reporting ratings on plans.

- Operating Load Rating – The plans shall have either the HS value of the operating rating if using LFR or the rating factor for the HL-93 if using LRFR. For LFR ratings, the rating should be rounded down to the nearest whole number. This rating shall be based on the current conditions of the bridge at the point when the construction is complete and shall not use the future wearing surface. See 6.2.2.3.4 for more information.

- Wisconsin Special Permit Vehicle – The plans shall also contain the results of the Wis-SPV analysis utilizing single-lane distribution and assuming that the vehicle is mixing with normal traffic and that the full dynamic load allowance is utilized. This rating shall be based on the current conditions of the bridge at the point when the construction is complete and shall not use the future wearing surface. The recorded rating for this is the total allowable vehicle weight rounded down to the nearest 10 kips. If the value exceeds 250 kips, limit the plan value to 250 kips. See 6.2.2.3.4 for more information.

45.9.4 Computer Software File Submittals

If analysis software is used to determine the load rating, the software input file shall be provided as a part of the submittal. The name of the analysis software and version should be noted on the Load Rating Summary form in the location provided.

45.9.5 Submittals for Bridges Rated Using Refined Analysis

Additional pages of documentation are required when performing a refined analysis. In addition to the Load Rating Summary Form, also submit the Refined Analysis Rating Form as shown in Figure 45.9-2.

45.9.6 Other Documentation Topics

Structures with Two Different Rating Methods

There may be situations where a given superstructure contains elements that were constructed at different times. In these situations, two different rating methods are used during the design/rating process. For example, a girder replacement or widening. In this case, the new girder(s) would be designed/rated using LRFR, while the existing girders would be rated using LFR. A Load Rating Summary Form shall be submitted for both new & existing structure analysis methods; controlling LRFR rating of the new superstructure components, and controlling LFR rating of the existing superstructure. Both sets of controlling rating values (new & existing) shall be noted on the plan set, as noted in 6.2.2.3.4.

BRIDGE DATA

Bridge Number:		Traffic Count:	
Region:		Traffic Year:	
Owner:		Truck Traffic %:	
Municipality:		Prior Inspection Date:	
Feature On:		Overburden Depth (in):	
Feature Under:		NBI Condition Ratings:	
Design Loading:		Deck: / Superstructure: / Substructure: / Culvert:	

STRUCTURE TYPE

Span #	Material	Configuration	Length (ft)

CONSTRUCTION HISTORY

Year	Work Performed

BRIDGE LOAD RATING SUMMARY

Rating Method:
- ☐ LRFR
- ☐ LFR
- ☐ ASR
- ☐ Field Evaluation / Eng. Judgment

Rating Vehicle: ☐ HL-93 ☐ HS20 ☐ User Defined (Describe in Remarks)

Ratings:	Controlling Element	Controlling Force Effect	LL Distribution Factor*
Inventory:			
Operating:			

* Enter "REFINED" if using a refined analysis. Submit Refined Analysis Rating Form.

Wisconsin Standard Permit Vehicle (Wis-SPV)	MVW (kips)	Controlling Element	Controlling Force Effect	LL Distribution Factor
Single Lane (w/o FWS)				
Multi Lane (w/o FWS)				

Posting Vehicles *	Vehicle GVW (Kips)	Operating Rating (Kips)	Controlling Element	Controlling Force Effect	LL Distribution Factor
Type 3	50				
Type 3S2	72				
Type 3-3	80				
SU4	54				
SU5	62				
SU6	69.5				
SU7	77.5				
PUP	98				
Semi	98				

Load Posting: ☐ Not Required ☐ Required:

* Posting Vehicle Analysis (when required per Wisconsin Bridge Manual, Chapter 45)

Computer Software Used (Name/Version):

Additional Remarks:

Load Rating Engineer

Name:

Date:

PE Stamp Here

Figure 45.9-1

Bridge Load Rating Summary Form

In Addition to this form, submit electronic analysis files (eg. .MDX, .bdb)

ANALYSIS FILE SUMMARY (FILL OUT FOR EACH ANALYSIS FILE SUBMITTED)

Analysis Type:	☐ Grid/Grillage ☐ Plate & Ecc. Beam ☐ 3D FEM ☐ Other *(describe below)*
Analysis Program:	☐ MDX ☐ AASHTOWare ☐ CSI Bridge ☐ LARSA ☐ Other
Program Version:	
File Name:	
File Description:	Describe the purpose of the file. Example: This file is used for the Wis-SPV rating using single lane distribution.
Analysis Assumptions:	Highlight key assumptions in modeling. (This section may be omitted if submitting MDX or AASHTOWare analysis files. This section may also be omitted if submitting separate document containing analysis assumptions and results). Example of things to include: a description of the finite element model, simplifications made to model, exceptions to original design plans, loads applied, how loads are applied (e.g. equally distributed to all girders), support conditions, composite/non-composite sections.
Summary of Results:	Summarize results. (This section may be omitted if submitting MDX or AASHTOWare analysis files. This section may also be omitted if submitting separate document containing analysis assumptions and results). Provide table of results for service load reactions, moment, shear, and/or stress output for members at 10th points (minimum) for the appropriate load cases. Provide a table of capacities at each 10th point, such that load ratings can be directly computed with appropriate load and/or resistance and impact factors. Provide example or typical calculations.

Figure 45.9-2

Refined Analysis Rating Form

For concrete box culverts, use Bridge Load Rating Summary form. All other bridge-length culverts shall use this form.

CULVERT DATA

Bridge Number:		**Traffic Count:**	
Owner:		**Traffic Year:**	
Municipality:		**Truck Traffic %:**	
Feature On:		**Prior Inspection Date**:**	
Design Loading*:		**Overburden Depth (in)**:**	
Design Overburden Depth (in)*:		**NBI Culvert Condition Rating**:**	

**For new culverts; if known for in-service culverts*

***For in-service culverts only*

STRUCTURE TYPE

Span #	Material	Configuration	Length (ft)

CONSTRUCTION HISTORY

Year	Work Performed

CULVERT LOAD RATING SUMMARY

Refer to Section 45.8 of the Wisconsin Bridge Manual for instructions on reporting load ratings.

Rating Method:

- ☐ LRFR
- ☐ LFR
- ☐ ASR
- ☐ Field Evaluation / Eng. Judgment

Rating Vehicle: ☐ HL-93 ☐ HS20 ☐ User Defined (Describe in Remarks)

RATINGS:

Inventory:	
Operating:	
MVW (Wisconsin SPV):	kips
Load Posting:	☐ Not Required ☐ Required (enter posting weight):

Additional Remarks:

Design or Load Rating Engineer

Name:

Date:

PE Stamp Here

Figure 45.9-3

Culvert Load Rating Summary Form

45.10 Load Postings

45.10.1 Overview

Legal-weight for vehicles travelling over bridges is determined by state-specific statutes, which are based in part on the Federal Bridge Formula. The Federal Bridge Formula is discussed in 45.2.5. When a bridge does not have the capacity to carry legal-weight traffic, more stringent load limits are placed on the bridge – a load posting. Currently in Wisconsin, load postings are based on gross vehicle weight; there is no additional consideration for number of axles or axle spacing. Load posting signage is discussed further in 45.10.4.

In order to remain open to traffic, a bridge should be capable of carrying a minimum gross live load weight of three tons at the Operating level. Bridges not capable of carrying a minimum gross live load weight of three tons at the Operating level must be closed. As stated in the **MBE [6A.8.1]** and **[6B.7.1]**, when deciding whether to close or post a bridge, the Owner should consider the character of traffic, the volume of traffic, the likelihood of overweight vehicles, and the enforceability of weight posting.

The owner of a bridge has the responsibility and authority to load post a bridge as required. The State Bridge Maintenance Engineer has the authority to post a bridge and must issue the approval to post any State bridge.

WisDOT policy items:

Consult the Bureau of Structures Rating Unit as soon as possible with any analysis that results in a load posting for any structure on the State or Local system.

45.10.2 Load Posting Live Loads

The live loads to be used in the rating formula for posting considerations are any of the three typical AASHTO Commercial Vehicles (Type 3, Type 3S2, Type 3-3) shown in Figure 45.10-1, any of the four AASHTO Specialized Hauling Vehicles (SHVs - SU4, SU5, SU6, SU7) shown in Figure 45.10-2, the WisDOT Specialized Annual Permit Vehicles shown in Figure 45.10-3, and the Wisconsin Standard Permit Vehicle shown in Figure 45.12-1.

The AASHTO Commercial Vehicles and Specialized Hauling Vehicles are modeled on actual in-service vehicle configurations. These vehicles comply with the provisions of the Federal Bridge Formula and can thus operate freely without permit; they are legal weight/configuration.

The WisDOT Specialized Annual Permit Vehicles are Wisconsin-specific vehicles. They represent vehicle configurations made legal in Wisconsin through the legislative process and current Wisconsin state statutes.

The Wisconsin Standard Permit Vehicle (Wis-SPV) is a configuration used internally by WisDOT to assist in the regulation of multi-trip (annual) permits. Multi-trip permits and the Wis-SPV are discussed in more detail in 45.11.2 and 45.12.

As stated in **MBE [6A.4.4.2.1a]**, for spans up to 200’, only the vehicle shall be considered present in the lane for positive moments. It is unnecessary to place more than one vehicle in a lane for spans up to 200’ because the load factors provided have been modeled for this possibility. For spans 200’ in length or greater, the AASHTO Type 3-3 truck multiplied by 0.75 shall be analyzed combined with a lane load as shown in Figure 45.10-4. The lane load shall be taken as 0.2 klf in each lane and shall only be applied to those portions of the span(s) where the loading effects add to the vehicle load effects.

Also, for negative moments and reactions at interior supports, a lane load of 0.2 klf combined with two AASHTO Type 3-3 trucks multiplied by 0.75 shall be used. The trucks should be heading in the same direction and should be separated by 30 feet as shown in Figure 45.10-4. There are no span length limitations for this negative moment requirement.

When the lane-type load model (see Figure 45.10-4) governs the load rating, the equivalent truck weight for use in calculating a safe load capacity for the bridge shall be taken as 80 kips as is specified in **MBE [6A.4.4.4]**.

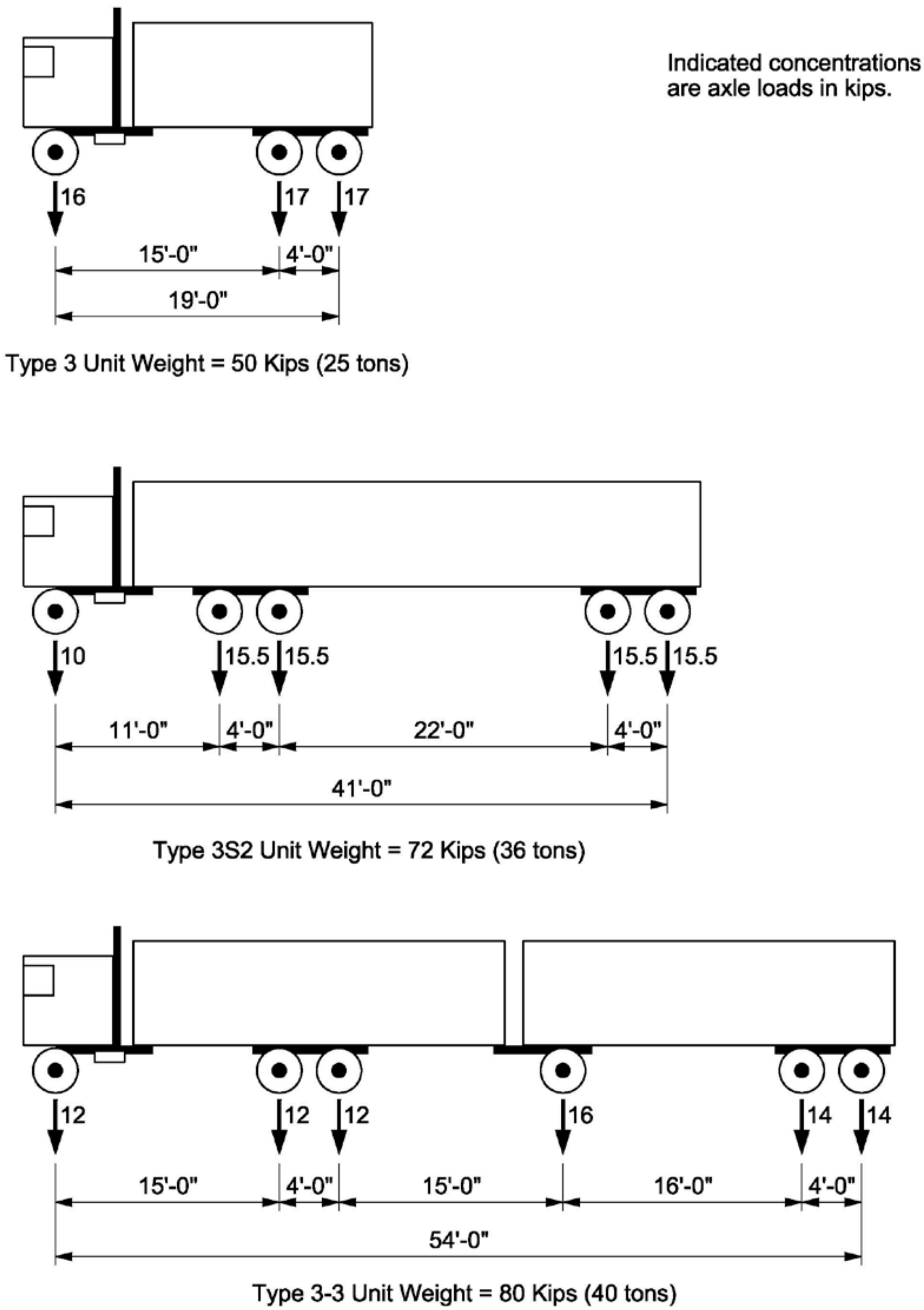


Figure 45.10-1
AASHTO Commercial Vehicles

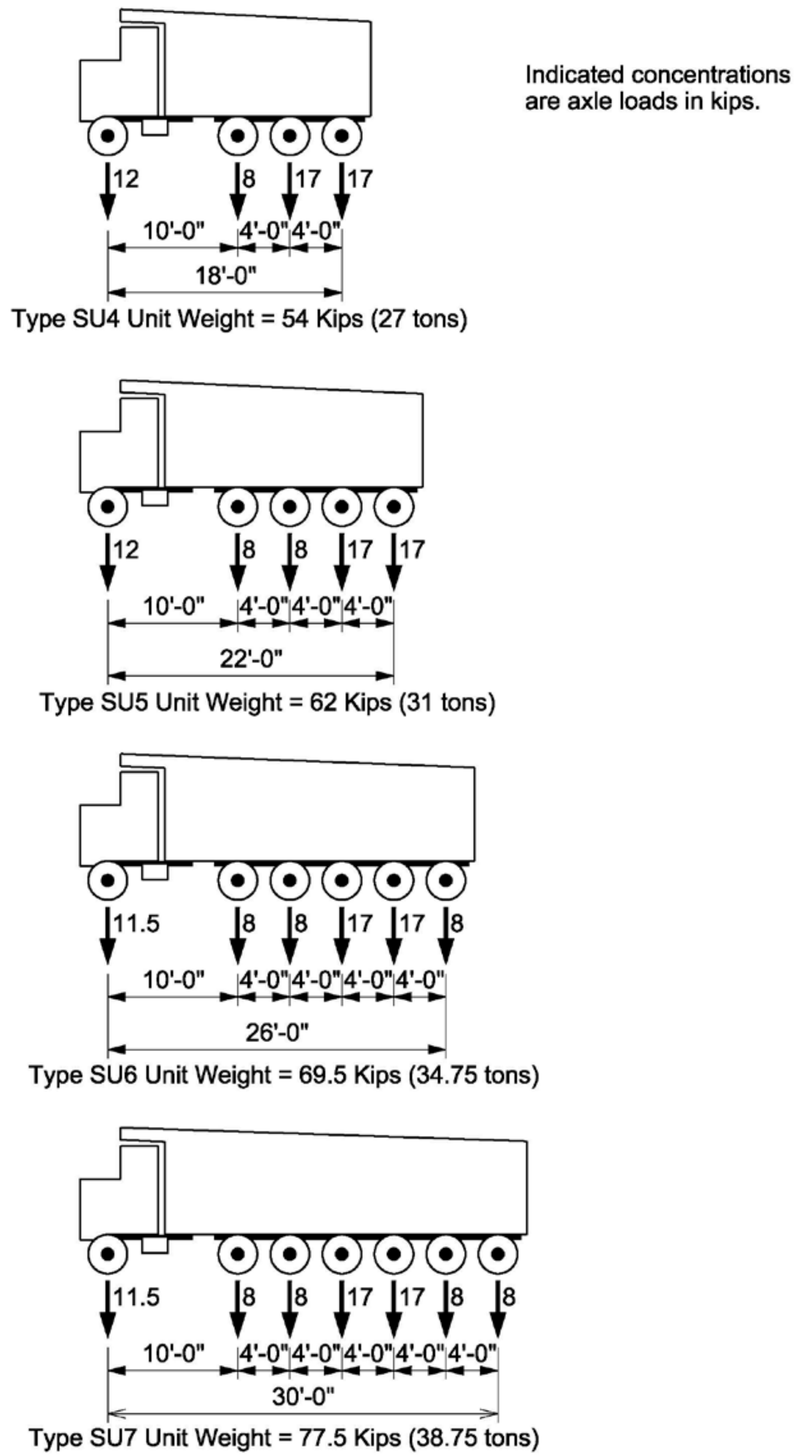


Figure 45.10-2
AASHTO Specialized Hauling Vehicles (SHVs)

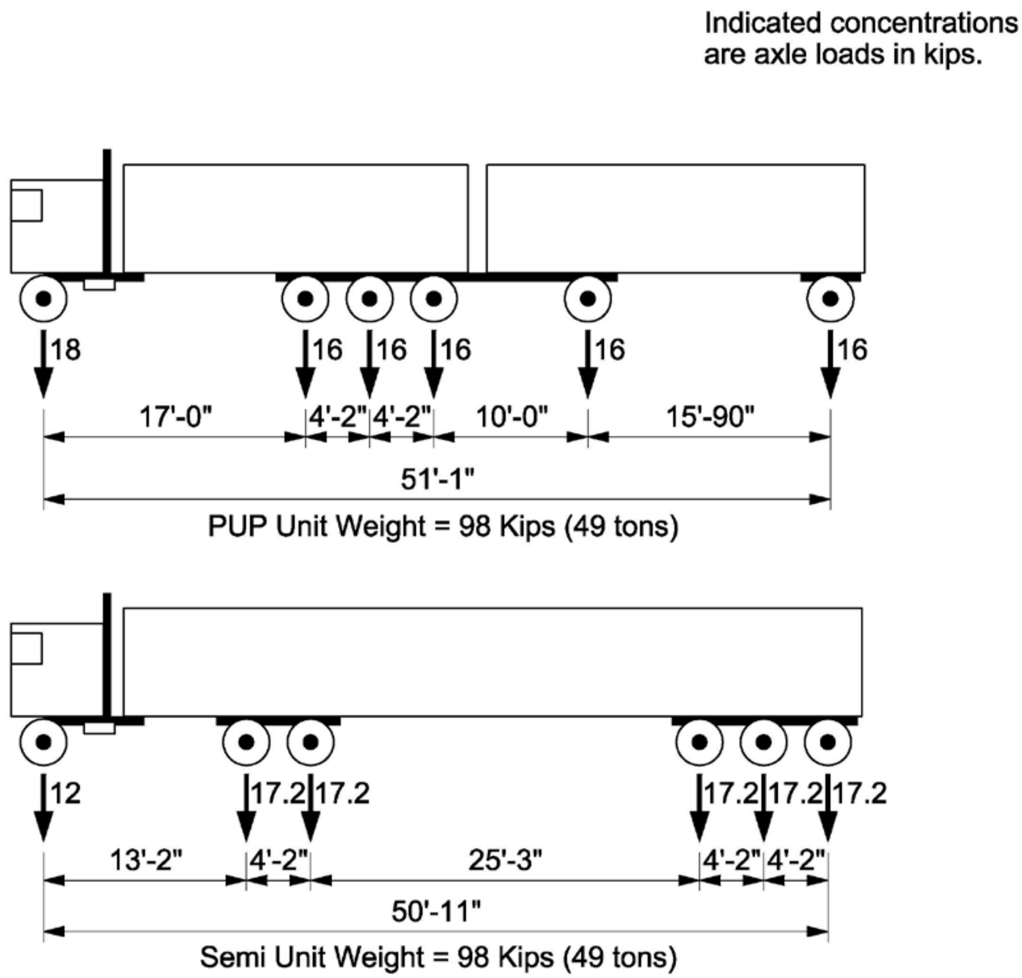
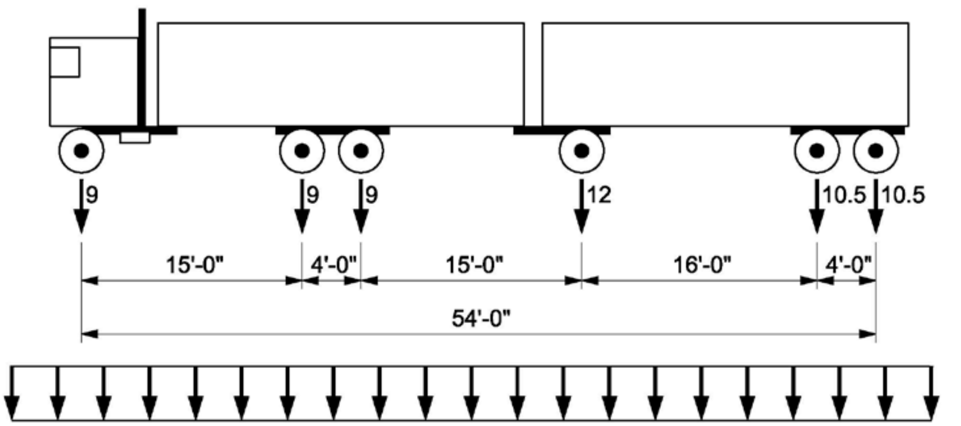
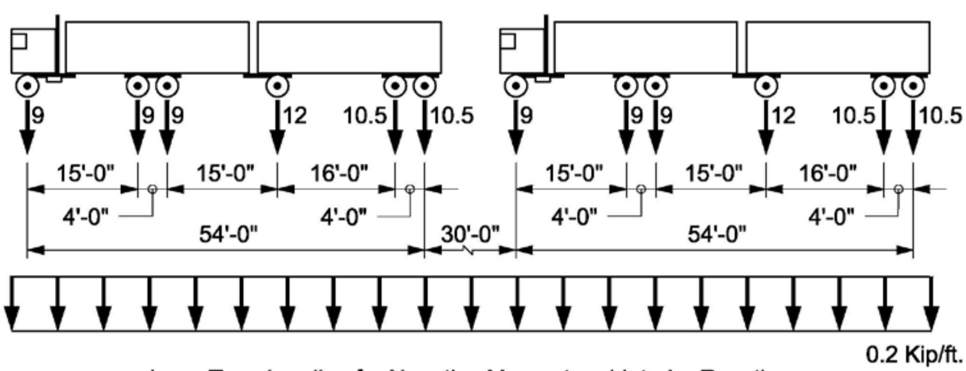


Figure 45.10-3

WisDOT Specialized Annual Permit Vehicles



Lane-Type Loading for Spans Greater Than 200 Ft.



Lane-Type Loading for Negative Moment and Interior Reaction.

Figure 45.10-4
Lane Type Legal Load Models

45.10.3 Load Posting Analysis

All posting vehicles shall be analyzed at the operating level. A load posting analysis is required when the calculated rating factor at operating level for a bridge is:

- Less than 1.0 for LRFR methodology.
- Less than 1.0 for LFR/ASR methodology; or
- Less than or equal to 1.2 for LFR/ASR methodology (SHV analysis only)
- Less than 1.25 for analysis of timber longitudinal slab superstructures

A load posting analysis is very similar to a load rating analysis, except the posting live loads noted in 45.10.2 are used instead of typical LFR or LRFR live loading.

If the calculated rating factor at operating is less than 1.0 for a given load posting vehicle, then the bridge shall be posted, with the exception of the Wis-SPV. For State Trunk Highway Bridges, current WisDOT policy is to post structures with a Wisconsin Standard Permit Vehicle (Wis-SPV) rating of 120 kips or less. If the RF ≥ 1.0 for a given vehicle at the operating level, then a posting is not required for that particular vehicle.

A bridge is posted for the lowest restricted weight limit of any of the standard posting vehicles. To calculate the capacity, in tons, on a bridge for a given posting vehicle utilizing LFR, multiply the rating factor by the gross vehicle weight in tons. To calculate the posting load for a bridge analyzed with LRFR, refer to 45.10.3.2.

45.10.3.1 Limit States for Load Posting Analysis

For LFR methodology, load posting analysis should consider strength-based limit states only.

For LRFR methodology, load posting analysis should consider strength-based limit states, but also some service-based limit states, per Table 45.3-1.

45.10.3.2 Legal Load Rating Load Posting Equation (LRFR)

When using the LRFR method and the operating rating factor (RF) calculated for each legal truck described above is greater than 1.0, the bridge does not need to be posted. When for any legal truck the RF is between 0.3 and 1.0, then the following equation should be used to establish the safe posting load for that vehicle (see **MBE [Equation 6A8.3-1]**):

$$\text{Posting} = \frac{W}{0.7}\left[(RF) - 0.3\right]$$

Where:

RF = Legal load rating factor

W = Weight of the rating vehicle

When the rating factor for any vehicle type falls below 0.3, then that vehicle type should not be allowed on the bridge. If necessary, the structure may need to be closed until it can be repaired, strengthened, or replaced. This formula is only valid for LRFR load posting calculations.

45.10.3.3 Distribution Factors for Load Posting Analysis

WisDOT policy items:

The AASHTO Commercial Vehicles and Specialized Hauling Vehicles shall be analyzed using a multi-lane distribution factor for bridge widths 18'-0" or larger. Single lane distribution factors are used for bridge widths less than 18'-0".

The WisDOT Specialized Annual Permit Vehicles shown in Figure 45.10-3 shall be analyzed using a single-lane distribution factor, regardless of bridge width.

The Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed for load postings using a multi-lane distribution factor for bridge widths 18'-0" or larger. Single lane distribution factors are used for bridge widths less than 18'-0".

45.10.4 Load Posting Signage

Current WisDOT policy is to post State bridges for a single gross weight, in tons. Bridges that cannot carry the maximum weight for the vehicles described in 45.10.2 at the operating level are posted with the standard sign shown in Figure 45.10-5. This sign shows the bridge capacity for the governing load posting vehicle, in tons. The sign should conform to the requirements of the *Wisconsin Manual for Uniform Traffic Control Devices (WMUTCD)*.

In the past, local bridges were occasionally posted with the signs shown in Figure 45.10-6 using the H20, Type 3 and Type 3S2 vehicles. The H20 represented the two-axle vehicle, the Type 3 represented the three-axle vehicle and the Type 3S2 represented the combination vehicle. This practice is not encouraged by WisDOT and is generally not allowed for State-owned structures, except with permission from the State Bridge Maintenance Engineer.

WEIGHT LIMIT 10 TONS	BRIDGE CLOSED

Figure 45.10-5
Standard Signs Used for Posting Bridges

WEIGHT LIMIT 2 AXLE VEHICLES 15 TONS 3 AXLE VEHICLES 20 TONS COMBINATION VEHICLES 30 TONS	WEIGHT LIMIT 2 AXLE VEHICLES 14 TONS 3 AXLE VEHICLES 18 TONS COMBINATION VEHICLES 28 TONS	WEIGHT LIMIT 2 AXLE VEHICLES 14 TONS 3 AXLE VEHICLES 18 TONS COMBINATION VEHICLES 28 TONS

Figure 45.10-6
Historic Load Posting Signs

45.11 Over-Weight Truck Permitting

45.11.1 Overview

Size and weight provisions for vehicles using the Wisconsin network of roads and bridges are specified in the Wisconsin Statutes, Chapter 348: Vehicles – Size, Weight and Load. Weight limits for legal-weight traffic and over-weight permit requirements are defined in detail in this chapter. The webpage for Chapter 348 is shown below.

http://docs.legis.wisconsin.gov/statutes/statutes/348

Over-weight permit requests are processed by the WisDOT Oversize Overweight (OSOW) Permit Unit in the Bureau of Highway Maintenance. The permit unit collaborates with the WisDOT Bureau of Structures Rating Unit to ensure that permit vehicles are safely routed on the Wisconsin inventory of bridges.

While the Wisconsin Statutes contain several industry-specific size and weight annual permits, in general, there are two permit types in Wisconsin: multi-trip (annual) permits and single-trip permits.

45.11.2 Multi-Trip (Annual) Permits

Multi-trip permits are granted for non-divisible loads such as machines, self-propelled vehicles, mobile homes, etc. They typically allow unlimited trips and are available for a range of three months to one year. The permit vehicle may mix with typical traffic and move at normal speeds. Multi-trip permits are required to adhere to road and bridge load postings and are subject to additional restrictions based on restricted bridge lists supplied by the WisDOT Bureau of Structures Rating Unit and published by the WisDOT OSOW Permit Unit. The restricted bridge lists are developed based on the analysis of the Wisconsin Standard Permit Vehicle (Wis-SPV). For more information on the Wis-SPV and required analysis, see 45.12. The carrier is responsible for their own routing, and are required to avoid these restrictions and load postings.

Vehicles applying for a multi-trip permit are limited to 170,000 pounds gross vehicle weight, plus additional restrictions on maximum length, width, height, and axle weights. Please refer to the WisDOT Oversize Overweight (OSOW) Permits website or the Wisconsin Statues (link above) for more information.

http://www.dot.wisconsin.gov/business/carriers/osowgeneral.htm

45.11.3 Single Trip Permits

Non-divisible loads which exceed the annual permit restrictions may be moved by the issuance of a single trip permit. When a single trip permit is issued, the applicant is required to indicate on the permit the origin and destination of the trip and the specific route that is to be used. A separate permit is required for access to local roads. Each single trip permit vehicle is individually analyzed by WisDOT for all state-owned structures that it encounters on the designated permit route.

Live load distribution for single trip permit vehicles is based on single lane distribution. This is used because these permit loads are infrequent and are likely the only heavy loads on the structure during the crossing. The analysis is performed at the operating level.

At the discretion of the engineer evaluating the single trip permit, the dynamic load allowance (or impact for LFR) may be neglected provided that the maximum vehicle speed can be reduced to 5 MPH prior to crossing the bridge and for the duration of the crossing.

In some cases, the truck may be escorted across the bridge with no other vehicles allowed on the bridge during the crossing. If this is the case, then the live load factor (LRFR analysis) can be reduced from 1.20 to 1.10 as shown in Table 45.3-3. It is recommended that the truck be centered on the bridge if it is being escorted with no other vehicles allowed on the bridge during the crossing.

Vehicles with non-standard axle gauges may also receive special consideration. This may be achieved by performing a more-rigorous analysis of a given bridge that takes into account the specific load configuration of the permit vehicle in question instead of using standard distribution factors that are based on standard-gauge axles. Alternatively, modifications may be made to the standard distribution factor in order to more accurately reflect how the load of the permit vehicle is transferred to the bridge superstructure. How non-standard gauge axles are evaluated is at the discretion of the engineer evaluating the permit.

45.12 Wisconsin Standard Permit Vehicle (Wis-SPV)

45.12.1 Background

The Wis-SPV configuration is shown in Figure 45.12-1. It is an 8-axle, 190,000lbs vehicle. It was developed through a Wisconsin research project that investigated the history of multi-trip permit configurations operating in Wisconsin. The Wis-SPV was designed to completely envelope the force effects of all multi-trip permit vehicles operating in Wisconsin and is used internally to help regulate multi-trip permits.

45.12.2 Analysis

- New Bridge Construction

For any new bridge design, the Wis-SPV shall be analyzed. The Wis-SPV shall be evaluated at the operating level. When performing this design check for the Wis-SPV, the vehicle shall be evaluated for single-lane distribution assuming that the vehicle is mixing with normal traffic and that the full dynamic load allowance is utilized. For this design rating, a future wearing surface shall be considered. Load distribution for this check is based on the interior strip or interior girder and the distribution factors given in Section 17.2.7, 17.2.8, or 18.4.5.1 where applicable. See also the WisDOT policy item in 45.3.7.8.1.

For LRFR, the Wis-SPV design check shall be a permit load rating and shall be evaluated for the limit states noted in Table 45.3-1 and Table 45.3-3.

The design engineer shall check to ensure the design has a RF > 1.0 (gross vehicle load of 190 kips) for the Wis-SPV. If the design is unable to meet this minimum capacity, the engineer is required to adjust the design until the bridge can safely handle a minimum gross vehicle load of 190 kips.

Results of the Wis-SPV analysis shall be reported per 45.9.

- Bridge Rehabilitation Projects

For rehabilitation design, analysis of the Wis-SPV shall be performed as described above for new bridge construction. All efforts should be made to obtain a RF > 1.0 (gross vehicle load of 190 kips) within the confines of the scope of the project. However, it is recognized that it may not be possible to increase the Wis-SPV rating without a significant change in scope of the project. In these cases, consult the Bureau of Structures Rating Unit for further direction.

Results of the Wis-SPV analysis shall be reported per 45.9.

- Existing (In-Service) Bridges

When performing a rating for an existing (in-service) bridge, analysis of the Wis-SPV shall be performed as described above for new bridge construction. In this case – where the bridge in question is being load rated but not altered in any way – the results of the Wis-SPV analysis need simply be reported as calculated per 45.9. If the results of this analysis produce a rating factor less than 1.0 (gross vehicle load less than 190 kips), notify the Bureau of Structures Rating Unit.

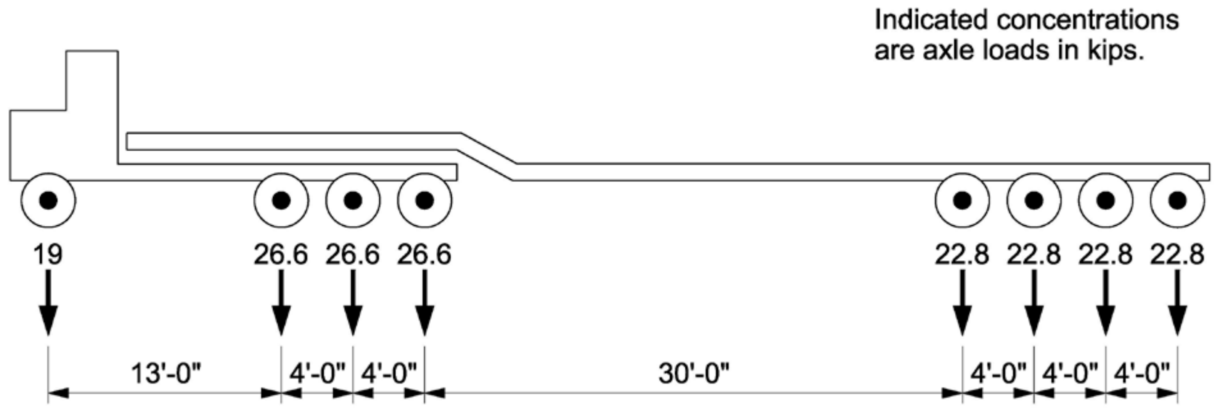


Figure 45.12-1
Wisconsin Standard Permit Vehicle (Wis-SPV)

45.13 References

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12. Wisconsin Statutes, Vol. 4, Chapter 348.

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