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5.1 Factors Governing Bridge Costs

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

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

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

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

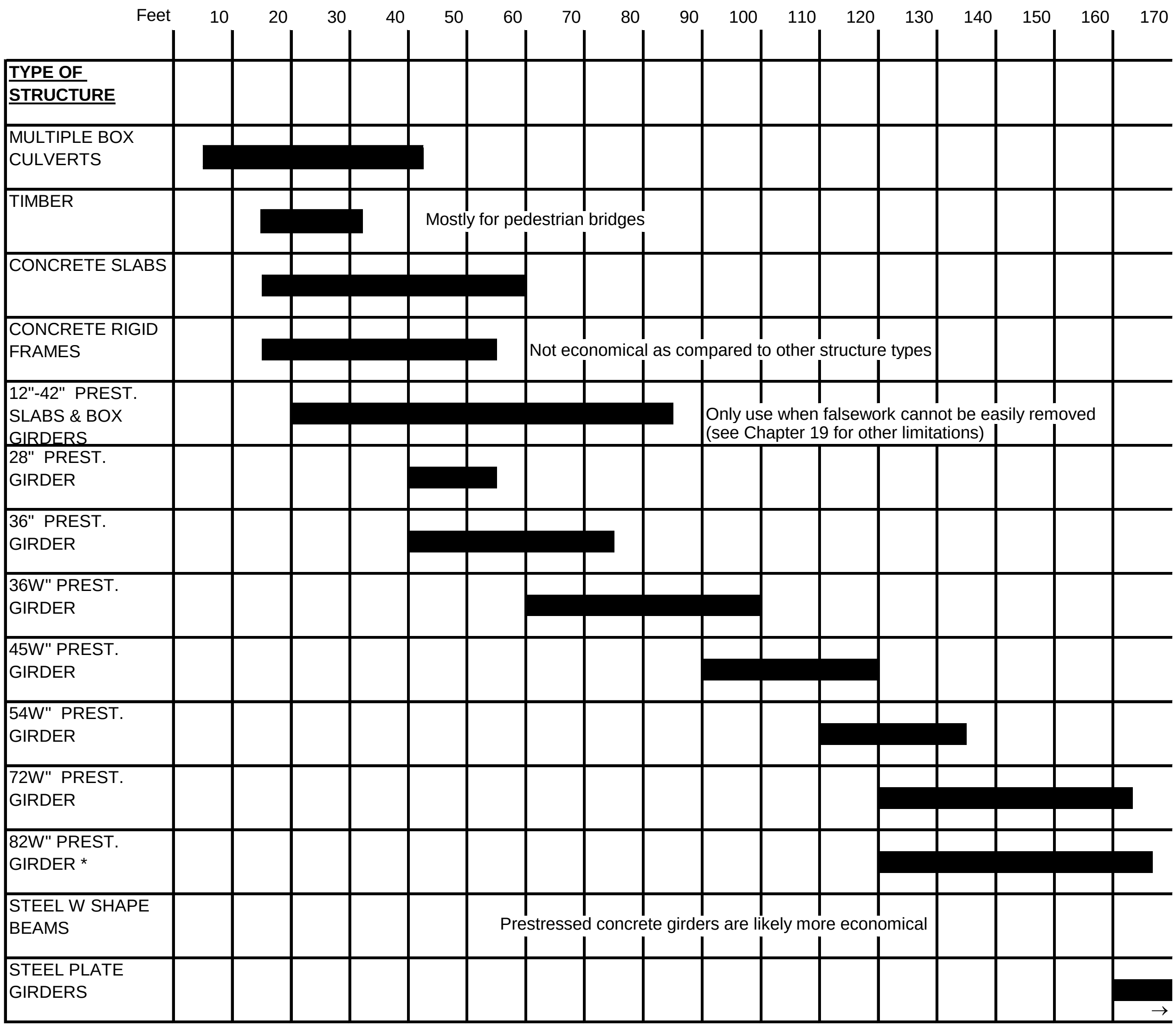
In these reports prestressed girder costs are grouped together because there is a small cost difference between girder sizes. Refer to unit costs. Concrete slab costs are also grouped together for this reason.

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

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

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

5.2 Economic Span Lengths



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

Figure 5.2-1
Economic Span Lengths

5.3 Contract Unit Bid Prices

Item No.	Bid Item	Unit	Cost
00000	Parapet Concrete Type LF & A (estimate)	LF	80.00
00000	Piling Steel Preboring	LF	122.33
00000	Preboring Cast-in-Place Concrete Piling	LF	25.00
210.0100	Backfill Structure	CY	18.63
303.0115	Pit Run	CY	13.01
311.0115	Breaker Run	CY	22.02
502.0100	Concrete Masonry Bridges	CY	601.43
502.1100	Concrete Masonry Seal	CY	383.65
502.2000	Compression Joint Sealer Preformed Elastomeric (width)	LF	25.67
502.3100	Expansion Device (structure) (LS)	LF	172.01
502.3110.S	Expansion Device Modular (structure) (LS)	LF	1,456.13
502.3200	Protective Surface Treatment	SY	2.36
502.3210.S	Pigmented Protective Surface Treatment	SY	6.34
502.6500	Protective Coating Clear	GAL	150.00
503.0128	Prestressed Girder Type I 28-Inch	LF	160.41
503.0136	Prestressed Girder Type I 36-Inch	LF	209.34
503.0137	Prestressed Girders Type I 36W-Inch	LF	162.63
503.0145	Prestressed Girder Type I 45-Inch	LF	160.00
503.0146	Prestressed Girders Type I 45W-Inch	LF	157.32
503.0154	Prestressed Girder Type I 54-Inch	LF	155.00
503.0155	Prestressed Girder Type I 54W-Inch	LF	149.78
503.0170	Prestressed Girder Type I 70-Inch	LF	--
503.0172	Prestressed Girders Type I 72W-Inch	LF	155.87
503.0182	Prestressed Girder Type I 82W-Inch	LF	--
504.0100	Concrete Masonry Culverts	CY	585.94
504.0500	Concrete Masonry Retaining Walls	CY	622.74
505.0405	Bar Steel Reinforcement HS Bridges	LB	0.82
505.0410	Bar Steel Reinforcement HS Culverts	LB	0.99
505.0415	Bar Steel Reinforcement HS Retaining Walls	LB	0.96
505.0605	Bar Steel Reinforcement HS Coated Bridges	LB	0.94
505.0610	Bar Steel Reinforcement HS Coated Culverts	LB	1.18
505.0615	Bar Steel Reinforcement HS Coated Retaining Walls	LB	0.99
505.0800.S	Bar Steel Reinforcement HS Stainless Structures	LB	2.29
506.0105	Structural Carbon Steel	LB	1.44
506.0605	Structural Steel HS	LB	1.14
506.2605	Bearing Pads Elastomeric Non-Laminated	EACH	69.49
506.2610	Bearing Pads Elastomeric Laminated	EACH	882.29
506.3005	Welded Stud Shear Connectors 7/8 x 4-Inch	EACH	3.06
506.3010	Welded Stud Shear Connectors 7/8 x 5-Inch	EACH	3.10
506.3015	Welded Stud Shear Connectors 7/8 x 6-Inch	EACH	5.52
506.3020	Welded Stud Shear Connectors 7/8 x 7-Inch	EACH	3.58
506.3025	Welded Stud Shear Connectors 7/8 x 8-Inch	EACH	5.00
506.4000	Steel Diaphragms (structure)	EACH	644.87
506.5000	Bearing Assemblies Fixed (structure)	EACH	1,213.46
506.6000	Bearing Assemblies Expansion (structure)	EACH	1,422.35
507.0200	Treated Lumber and Timber	MBM	5,306.70
508.1600	Piling Treated Timber Delivered	LF	33.75

511.1200	Temporary Shoring (structure)	SF	26.02
512.1000	Piling Steel Sheet Temporary	SF	13.78
513.4050	Railing Tubular Type F (structure) (LS)	LF	136.05
513.4052/4053	Railing Tubular Type F- (4 or 5) Modified (structure) (LS)	LF	128.44
513.4055	Railing Tubular Type H (structure) (LS)	LF	97.64
513.4060	Railing Tubular Type M (structure) (LS)	LF	206.48
513.4065	Railing Tubular Type PF (structure) (LS)	LF	--
513.4090	Railing Tubular Screening (structure) (LS)	LF	157.07
513.7005	Railing Steel Type C1 (structure) (LS)	LF	93.45
513.7010	Railing Steel Type C2 (structure) (LS)	LF	112.58
513.7015	Railing Steel Type C3 (structure) (LS)	LF	90.83
513.7020	Railing Steel Type C4 (structure) (LS)	LF	151.54
513.7025	Railing Steel Type C5 (structure) (LS)	LF	206.95
513.7030	Railing Steel Type C6 (structure) (LS)	LF	111.49
513.7050	Railing Type W (structure) (LS)	LF	145.97
514.0445	Floor Drains Type GC	EACH	1,806.67
514.2608	Downspout 8-Inch	LF	200.82
514.2625	Downspout 6-Inch	LF	164.48
516.0500	Rubberized Membrane Waterproofing	SY	24.52
517.1010.S	Concrete Staining (structure)	SF	1.09
517.1015.S	Concrete Staining Multi-Color (structure)	SF	2.90
517.1050.S	Architectural Surface Treatment (structure)	SF	4.36
550.0010	Preboring Unconsolidated Materials	LF	55.74
550.0020	Preboring Rock or Consolidated Materials	LF	69.22
550.0500	Pile Points	EACH	127.65
550.1100	Piling Steel HP 10-Inch x 42 LB	LF	34.80
550.1120	Piling Steel HP 12-Inch x 53 LB	LF	36.40
550.1125	Piling Steel HP 12-Inch x 74 LB	LF	--
550.1140	Piling Steel HP 14-Inch x 73 LB	LF	51.86
550.2102	Piling CIP Concrete 10 ¾ x 0.219-Inch	LF	29.15
550.2104	Piling CIP Concrete 10 ¾ x 0.25-Inch	LF	36.34
550.2106	Piling CIP Concrete 10 ¾ x 0.365-Inch	LF	34.62
550.2108	Piling CIP Concrete 10 ¾ x 0.5-Inch	LF	--
550.2122	Piling CIP Concrete 12 ¾ x 0.219-Inch	LF	--
550.2124	Piling CIP Concrete 12 ¾ x 0.25-Inch	LF	35.82
550.2126	Piling CIP Concrete 12 ¾ x 0.375-Inch	LF	46.94
550.2128	Piling CIP Concrete 12 ¾ x 0.5-Inch	LF	46.20
550.2142	Piling CIP Concrete 14 x 0.219-Inch	LF	--
550.2144	Piling CIP Concrete 14 x 0.25-Inch	LF	52.00
550.2146	Piling CIP Concrete 14 x 0.375-Inch	LF	42.00
550.2148	Piling CIP Concrete 14 x 0.5-Inch	LF	70.00
550.2162	Piling CIP Concrete 16 x 0.219-Inch	LF	--
550.2164	Piling CIP Concrete 16 x 0.25-Inch	LF	--
550.2166	Piling CIP Concrete 16 x 0.375-Inch	LF	--
550.2168	Piling CIP Concrete 16 x 0.5-Inch	LF	57.00
604.0400	Slope Paving Concrete	SY	59.86
604.0500	Slope Paving Crushed Aggregate	SY	22.91
604.0600	Slope Paving Select Crushed Material	SY	18.75
606.0100	Riprap Light	CY	59.52
606.0200	Riprap Medium	CY	51.30

606.0300	Riprap Heavy	CY	48.94
606.0500	Grouted Riprap Light	CY	--
606.0600	Grouted Riprap Medium	CY	--
606.0700	Grouted Riprap Heavy	CY	90.00
612.0106	Pipe Underdrain 6-Inch	LF	7.56
612.0206	Pipe Underdrain Unperforated 6-Inch	LF	7.19
612.0406	Pipe Underdrain Wrapped 6-Inch	LF	7.21
614.0150	Anchor Assemblies for Steel Plate Beam Guard	Each	125.22
616.0204	Fence Chain Link 4-FT	LF	46.02
616.0205	Fence Chain Link 5-FT	LF	22.80
616.0206	Fence Chain Link 6-FT	LF	35.64
616.0208	Fence Chain Link 8-FT	LF	68.51
645.0105	Geotextile Fabric Type C	SY	4.81
645.0111	Geotextile Fabric Type DF (Schedule A)	SY	5.13
645.0120	Geotextile Fabric Type HR	SY	3.06
652.0125	Conduit Rigid Metallic 2-Inch	LF	21.36
652.0135	Conduit Rigid Metallic 3-Inch	LF	29.82
652.0225	Conduit Rigid Nonmetallic Schedule 40 2-Inch	LF	4.40
652.0235	Conduit Rigid Nonmetallic Schedule 40 3-Inch	LF	5.89
657.6005.S	Anchor Assemblies Light Poles	Each	555.20
SPV.0035	HPC Masonry Superstructure	CY	528.60
SPV.0105	Parapet Concrete Type TX (LS) (estimate)	LF	179.52
SPV.0165	Anti-Graffiti Coating	SF	1.07
SPV.0180	Protective Polymer Coating	SY	--

Table 5.3-1

Contract Unit Bid Prices for New Structures

Item No.	Bid Item	Unit	Cost
455.0___	Asphaltic Material ________	TON	100.00
460.1___	HMA Pavement Type ____	TON	43.54
502.5002	Masonry Anchors Type L No. 4 Bars	EACH	20.11
502.5005	Masonry Anchors Type L No. 5 Bars	EACH	19.89
502.5010	Masonry Anchors Type L No. 6 Bars	EACH	19.43
502.5015	Masonry Anchors Type L No. 7 Bars	EACH	25.09
502.5020	Masonry Anchors Type L No. 8 Bars	EACH	31.57
502.5025	Masonry Anchors Type L No. 9 Bars	EACH	30.00
502.6102	Masonry Anchors Type S 1/2-Inch	EACH	14.35
502.6105	Masonry Anchors Type S 5/8-Inch	EACH	16.42
502.6110	Masonry Anchors Type S 3/4-Inch	EACH	19.19
502.6115	Masonry Anchors Type S 7/8-Inch	EACH	--
502.6120	Masonry Anchors Type S 1-Inch	EACH	--
505.0904	Bar Couplers No. 4	EACH	35.43
505.0905	Bar Couplers No. 5	EACH	31.44
505.0906	Bar Couplers No. 6	EACH	35.69
505.0907	Bar Couplers No. 7	EACH	55.00
505.0908	Bar Couplers No. 8	EACH	58.11
505.0909	Bar Couplers No. 9	EACH	90.14
509.0301	Preparation Decks Type 1	SY	69.96
509.0302	Preparation Decks Type 2	SY	107.66
509.0500	Cleaning Decks	SY	13.27
509.1000	Joint Repair	SY	750.34
509.1200	Curb Repair	LF	43.42
509.1500	Concrete Surface Repair	SF	92.65
509.2000	Full-Depth Deck Repair	SY	276.34
509.2500	Concrete Masonry Overlay Decks	CY	666.23
509.5100.S	Polymer Overlay	SY	20.57

Table 5.3-2

Contract Unit Bid Prices for Rehab Structures

5.4 Bid Letting Cost Data

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

5.4.1 2010 Year End Structure Costs

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

Table 5.4-1
Stream Crossing Structure

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

Table 5.4-2
Grade Separation Structures

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

Table 5.4-3
Box Culverts

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

Table 5.4-4
Pre-Fab Pedestrian Bridge

Pedestrian Bridges	Cost per Sq. Ft.
4	179.56

Table 5.4-5
Pedestrian Bridges

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

Table 5.4-6
Retaining Walls

5.4.2 2011 Year End Structure Costs

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

Table 5.4-7
Stream Crossing Structures

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

Table 5.4-8
Grade Separation Structures

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

Table 5.4-9
Box Culverts

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

Table 5.4-10
Railroad Bridges

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

Table 5.4-11
Retaining Walls

5.4.3 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 5.4-12
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 5.4-13
Grade Separation Structures

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

Table 5.4-14
Box Culverts

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

Table 5.4-15
Pre-Fab Pedestrian Bridge

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

Table 5.4-16
Pedestrian Bridges

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

Table 5.4-17
Buried Slab Bridges

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

Table 5.4-18
Retaining Walls

5.4.4 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 5.4-19
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
Trapezoid Box	7	571,326	98,535,301	116.21	172.47

Table 5.4-20
Grade Separation Structures

Box Culverts	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 5.4-21
Box Culverts

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

Table 5.4-22
Pre-Fab Pedestrian Bridge

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

Table 5.4-23
Pedestrian Bridges

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

Table 5.4-24
Buried Slab Bridges

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

Table 5.4-25
Railroad Bridges

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

Table 5.4-26
Inverted T Bridges

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

Table 5.4-27
Retaining Walls

5.4.5 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 5.4-28
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-Beam	1	2,078	596,712	82.99	287.16
Pedestrian Bridges	3	35,591	7,436,429	--	208.94
Trapezoidal Steel Box Girders	1	59,128	9,007,289	121.00	152.34

Table 5.4-29
Grade Separation Structures

Box Culverts	No. of Culverts	Cost per Lin. Ft.
Single Cell	12	3,645.06
Twin Cell	4	2,584.21
Triple Cell	1	2,928.40
Triple Pipe	1	1,539.41

Table 5.4-30
Box Culverts

Retaining Walls	No. of Bridges	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
Wired Faced MSE Wall	2	3,747	537,173	143.36
Secant Pile Wall	1	68,326	7,488,658	109.60
Soldier Pile Wall	9	33,927	4,470,908	131.78
Steel Sheet Pile Wall	2	3,495	159,798	45.72

Table 5.4-31
Retaining Walls

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

Table 5.4-32
Noise Walls

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

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

4. List design data including:

   Material Properties:

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

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

Foundations

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

Ratings (Plans Including Ratings that have been Changed)

Live Load:

Design Loading: HL-93

Inventory Rating Factor: RF = X.XX

Operating Rating Factor: RF = X.XX

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

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

Ratings (Plans Including Ratings that have not been Changed)

Live Load:

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

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

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

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

Hydraulic Data

Base Flood

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

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

- Overtopping Frequency
- Overtopping Elevation
- Overtopping Discharge

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

6.2.2.4 Utilities

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

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

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

It is the general policy to not place utilities on the structure. The Utilities & Access Management Unit approves all utility applications and determines whether utilities are placed total length of 20 feet is to be bid under item Concrete Masonry Culverts. As another example, a bridge having a length less than 20 feet would be given a "C" number; however, the concrete bid item is Concrete Masonry Bridges.

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

9. General Notes

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

10. List of Drawings

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

11. Bench Marks

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

12. Title Block

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

13. Professional Seal

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

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

6.3.2.1.1 Plan Notes for New Bridge Construction

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

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

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

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

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

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

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

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

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

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

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

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

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

14. The quantity for Backfill Structure, bid item 210.0100, is calculated based on the applicable Figures 12.6-1 and 12.6-2 in the Wisconsin Department of Transportation Bridge Manual.

6.3.2.1.2 Plan Notes for Bridge Rehabilitation

WisDOT policy item:

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

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

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

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

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

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

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

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

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

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

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

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

6.3.2.2 Subsurface Exploration

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

1. Plan View

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

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

2. Elevation

   Show a centerline profile of existing ground elevation.

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

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

6.3.2.3 Abutments

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

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

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

1. Plan View

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

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

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

2. Elevation

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

    b. Give vertical dimension of wing.

3. Wing Elevation

4. Body Section

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

5. Wing Sections

6. Bar Steel Listing and Detail

Use the following views where necessary:

7. Pile Plan & Splice Detail

8. View Showing Limits of Excavation and Backfill

9. Special Details for Utilities

10. Drainage Details

6.3.2.4 Piers

Use as many sheets as necessary to show all details clearly. Each substructure unit should have its own plan sheet(s).

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

1. Plan View

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

2. Elevation

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

3. Footing Plan

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

4. Bar Steel Listing and Details

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

6. Cross Section thru Column and Pier Cap

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

6.3.2.5 Superstructure

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

6.3.2.5.1 All Structures

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

2. For girder bridges:
   Show the total dead load deflections, including composite dead load (without future wearing surface) acting on the composite section, at tenth points of each span. Distribute the composite dead load evenly to all girders and provide one deflection value for a typical interior girder. Chapter 17 – Superstructure-General illustrates three load cases for exterior girder design with raised sidewalks, cases that provide a conservative envelope to ensure adequate girder capacity. However, the above composite dead load distribution should be used for deflection purposes. For prestressed concrete girders, the dead load deflection reported does not include the weight of the girder. See Chapter 24 – Steel Girder Structures for camber and blocking, top of steel elevation and deflection reporting criteria.

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

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

   Deflection and camber values are to be reported to the nearest 0.1 inch, for all girder and slab superstructures.

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

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

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, sign bridges, pedestrian bridges, and erosion control structures are to be detailed with the same requirements as previously mentioned.

6.3.3.4 Standard Drawings

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

6.3.3.5 Insert Sheets

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

6.3.3.6 Change Orders and Maintenance Work

These plans are drawn on full size sheets. 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.

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. Bench mark disks shall be shown on all 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. Locate the name plate on the roadway side of the first right wing or railing traveling in the highway cardinal directions of North or East. For type “F”, “NY”, “W”, “M” or timber railings, name plate to be located on wing. For all other railing types, name plate to be located on inside face of railing.

6.3.4 Checking Plans

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

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

The Checker makes an independent check of the Bill of Bars list to ensure the Plan Preparer has not omitted any bars when determining the quantity of bar steel.

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

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

After the plans are completed, the items in the project folder are separated into the following groups by the Structures Design Engineer:

6.3.4.1 Items requiring a PDF copy for the Project Records (Group A) – Paper Copies to be Destroyed when Construction is Completed.

1. QC/QA sign-off sheet

2. Design computations

3. Quantity computations

4. Bridge Special Provisions and STSP's (only those STSP's requiring specific blanks to be filled in or contain project specific information)

5. Final Structure Survey Report form (not including photos, cross-sections, project location maps, etc.)

6. Final Geotechnical Report

6.3.4.2 Additional Items to be Destroyed When Construction is Completed (Group B)

1. Miscellaneous correspondence and transmittal letters

2. Preliminary drawings and computations

3. Prints of soil borings and plan profile sheets

4. Shop steel quantity computations*

5. Design checker computations

6. Designer computations and computer runs of non-complex structures on non state maintained structures

7. Layout sheets

8. Elevation runs and bridge geometrics

9. Falsework plans*

10. Miscellaneous Test Report

11. Photographs of bridge rehabs

* These items are added to the packet during construction.

6.3.4.3 Items to be Destroyed when Plans are Completed (Group C)

1. All "void" material

2. All copies except one of preliminary drawings

3. Extra copies of plan and profile sheets

4. Preliminary computer design runs

Note that lists for Group A, B & C are not intended to be all inclusive, but serve as starting points for categorizing design material. Items in Group A & B should be labeled separately. Items in Group C should be discarded.

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

1. Structure Inventory Form (Available on DOTNET) - New Structures File – Data for this form is completed by the Structural Design Engineer. It is E-submitted to the Structures Development Section for entry into the File.

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

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

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

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

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

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

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

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

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

11. Inspection Reports - A certified bridge inspector enters the initial inspection data into HSI.

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

Table 6.3-2
Various Inspection Reports

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

6.4 Computation of Quantities

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

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

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

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

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

6.4.1 Excavation for Structures Bridges (Structure)

This is a lump sum bid item. The limits of excavation are shown in the chapter in the manual which pertains to the structural item, abutments, piers, retaining walls, box culverts, etc. If the excavation is required for the roadway, the work may be covered under Excavation Common.

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

6.4.2 Backfill Granular or Backfill Structure

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

6.4.3 Concrete Masonry Bridges

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

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

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

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

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

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

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

6.4.6 Bar Steel Reinforcement HS Stainless Bridges

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

6.4.7 Structural Steel Carbon or Structural Steel HS

See 24.2.4.

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

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

6.4.9 Piling Test Treated Timber (Structure)

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

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

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

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

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

6.4.11 Preboring CIP Concrete Piling or Steel Piling

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

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

Record the type, quantity is a Lump Sum.

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

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

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

Record this quantity to the nearest 5 cubic yards.

6.4.15 Pile Points

When recommended in soils report. Bid as each.

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

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

6.4.17 Cofferdams (Structure)

Lump Sum

6.4.18 Rubberized Membrane Waterproofing

Record the quantity to the nearest square yard.

6.4.19 Expansion Device (Structure)

Record this quantity in lump sum.

6.4.20 Electrical Work

Refer to Standard Construction Specifications for bid items.

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

Record this quantity in feet.

6.4.22 Preparation Decks Type 1 or Preparation Decks Type 2

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

6.4.23 Cleaning Decks

Record this quantity to the nearest square yard.

6.4.24 Joint Repair

Record this quantity to the nearest square yard.

6.4.25 Concrete Surface Repair

Record this quantity to the nearest square foot.

6.4.26 Full-Depth Deck Repair

Record this quantity to the nearest square yard.

6.4.27 Concrete Masonry Overlay Decks

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

6.4.28 Removing Old Structure STA. XX + XX.XX

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

6.4.29 Anchor Assemblies for Steel Plate Beam Guard

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

6.4.30 Steel Diaphragms (Structure)

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

6.4.31 Welded Stud Shear Connectors X -Inch

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

6.4.32 Concrete Masonry Seal

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

6.4.33 Geotextile Fabric Type

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

6.4.34 Masonry Anchors Type L No. Bars

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

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

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

6.4.36 Piling Steel Sheet Temporary

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

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

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

6.4.37 Temporary Shoring

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

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

6.4.38 Concrete Masonry Deck Patching

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

6.4.39 Sawing Pavement Deck Preparation Areas

Use 10 lineal feet per SY of Preparation Decks.

6.4.40 Removing Bearings

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

6.4.41 Ice Hot Weather Concreting

Used to provide mechanism for payment of ice during hot weather concreting operations. See FDM 14-10-10.10 for bid item usage guidance and quantity calculation guidance. Bid as LB and round to the nearest 5 lbs.

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

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

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

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

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

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

6.5.1 Approvals, Distribution, and Work Flow

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

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7.1 Introduction

Disclaimer:

This chapter is in the early stages of development. The information is limited and will develop over time. The intent of this chapter is to provide guidance to designers, but is far from all-inclusive.

The purpose of the Accelerated Bridge Construction (ABC) Chapter is to provide guidance for the planning and implementation of projects that may benefit from the application of rapid bridge construction technologies and methods. This chapter was prepared to provide planners and engineers with a basic understanding of different ABC methods available, help guide project specific selection of ABC methods, and to encourage the use of the ABC methods described in this chapter.

7.1.1 WisDOT ABC Initiative

The Department's mission is to provide leadership in the development and operation of a safe and efficient transportation system. One of our values relates to Improvement - Finding innovative and visionary ways to provide better products and services and measure our success. The application of Accelerated Bridge Construction (ABC) is consistent with our Mission and Values in promoting efficient development and operation of the transportation system through innovative bridge construction techniques that better serve the public. This service may manifest as safer projects with shorter and less disruptive impacts to the traveling public, and potential cost savings.

WisDOT is following the Federal Highway Administration's (FHWA) Every Day Counts initiative "aimed at shortening project delivery, enhancing the safety of our roadways, and protecting the environment." Two of the five major methods that the FHWA has emphasized as accelerating technologies are Prefabricated Bridge Elements and Systems (PBES) and Geosynthetic Reinforced Soil – Integrated Bridge Systems (GRS-IBS). These accelerating technologies are incorporated in the following sections in this chapter, namely: Prefabricated Bridge Elements, Geosynthetic Reinforced Soil – Integrated Bridge Systems (GRS-IBS), Self Propelled Modular Transporters (SPMTs) and Lateral Sliding (both SPMTs and Lateral Sliding are classified as Prefabricated Bridge Systems). WisDOT has had success using GRS-IBS and Prefabricated Bridge Elements, and is always looking for new technologies to improve construction and reduce impacts to traffic. For more information on the Every Day Counts Initiative, refer to www.fhwa.dot.gov/everydaycounts.

7.1.2 ABC Overview

In essence, ABC uses different methods of project delivery and construction to reduce the project schedule, on-site construction time, and public impact. With the ever increasing demand on transportation infrastructure, and the number of bridges that are approaching the end of their service lives, the need for ABC becomes more apparent.

Three main benefits of using ABC methods include minimized impact to traffic, increased safety during construction, and minimized impacts in environmentally sensitive areas. Where conventional bridge construction takes months or years, a bridge utilizing ABC may be placed in a matter of weeks, days, or even a few hours depending on the methods used.

ABC methods are generally safer than conventional construction methods because much of the construction can be done offsite, away from traffic. Quality can also be improved because the construction is often completed in a more controlled environment compared to on-site conditions. On the other hand, as with the implementation of all new technologies, the use of ABC comes with challenges that need to be overcome on a project-specific basis.

Oftentimes accelerating the schedule increases the cost of the project. This increased project delivery cost can be offset by reductions in road user costs. In some states, it has been shown that a high percentage of the public approves the use of ABC knowing that the cost can be significantly higher.

WisDOT policy item:

Prior to the implementation of ABC methods on a project, contact the Bureau of Structures Development Section Chief for discussion, resources, and approval.

7.1.3 Accelerated Bridge Construction Technology

Acronym/Term	Definition
ABC (Accelerated Bridge Construction)	Bridge construction methods that use innovative planning, design, materials, and construction techniques in a safe and cost-effective manner to reduce the onsite construction time that occurs when building new bridges or replacing and rehabilitating existing bridges.
AC (Alternative Contracting)	Nontraditional project delivery systems, bidding practices, and specifications that may be used to reduce life-cycle costs, improve quality, and accelerate the delivery of construction projects.
BSA (Bridge Staging Area)	Location where a bridge is constructed near the final location for the bridge, where the traveling public is not affected. The bridge can be moved from the staging area to the final location with SPMTs or by sliding.
CM/GC (Construction Manager/General Contractor)	Hybrid of the DBB and D/B processes that allows the owner to remain active in the design process, while the risk is still taken by the general contractor. This method is not an option for WisDOT administered projects.
D/B (Design/Build)	Accelerated project delivery method where one entity (the “designer-builder”) assumes responsibility for both the design and construction of a project. This method is not an option for WisDOT administered projects.
DBB (Design-Bid-Build)	Traditional project delivery method where the owner contracts out the design and construction of a project to two different entities.

EDC (Every Day Counts)	Initiative put forth by FHWA designed to identify and deploy innovation aimed at shortening project delivery, enhancing the safety of our roadways, and protecting the environment.
GRS-IBS (Geosynthetic Reinforced Soil – Integrated Bridge System)	An ABC technology that uses alternating layers of compacted granular fill material and fabric sheets of geotextile reinforcement to provide support for the bridge in place of a traditional abutment.
LBDB (Low Bid Design Build)	A type of D/B where the design and construction service is bundled into a single contract awarded to the lowest competent and responsible bidder.
PBES (Prefabricated Bridge Elements and Systems)	Structural components of a bridge or bridge system that are constructed offsite, or near-site of a bridge that reduce the onsite construction time and impact to the traveling public relative to conventional construction methods.
Pick Points	Locations where the SPMTs will lift and carry the bridge.
Program Initiative	The use of ABC methods to facilitate research, investigate technology, develop familiarity, or address other stakeholder needs.
Road User Costs	Costs pertaining to a project alternative borne by motorists and the community at-large as a result of work zone activity. (FDM 11-50-32)
SPMTs (Self Propelled Modular Transporters)	Remote-controlled, multi-axle platform vehicles capable of transporting several thousand tons of weight.
Stroke	Distance an SPMT can raise or lower its platform.
TMP (Transportation Management Plan)	A set of coordinated transportation management strategies that describes how they will be used to manage work zone impacts of a road project. (FDM 11-50-5)
TP (Travel Path)	Course that the SPMTs travel to carry the completed structure from the staging area to the final location.

Table 7.1-1
ABC Terminology

7.1.4 ABC Methods

7.1.4.1 Prefabricated Bridge Elements

Prefabricated bridge elements are a commonly used ABC method and can be incorporated into most bridge projects as a form of accelerated construction. Concrete bridge elements are prefabricated, transported to the construction site, placed in the final location, and tied into the structure. An entire bridge can be composed of prefabricated elements, or single bridge elements can be prefabricated as the need arises. Prefabricated bridge elements can also be used in combination with other accelerated bridge construction methods. Commonly used prefabricated bridge elements are prestressed concrete girders (including I-girders, adjacent inverted T-beams, and boxes), full depth and partial depth deck panels, abutments, pier caps, pier columns, and footings, as well as precast three-sided and four-sided box culverts.

For all prefabricated bridge elements, shop drawings shall be submitted by email to the Bureau of Structures Development Section Chief.

Figure 7.1-1
Prefabricated Pier Cap

Figure 7.1-2
Prefabricated Abutment

Prefabricated bridge elements are used to mitigate the on-site time required for concrete forming, rebar tying and concrete curing, saving weeks to months of construction time. Deck beam elements eliminate conventional onsite deck forming activities. To reduce onsite deck forming operations, deck beam elements are typically placed in an abutting manner. Prefabricated elements are often of higher quality than conventional field-constructed elements, because the concrete is cast and cured in a controlled environment. The elements are often connected using high strength grout, and post-tensioning or pretensioning. Because some previous prefabricated bridge element connections have had problems, close attention should be given to these connections.

7.1.4.1.1 Precast Piers

Precast concrete piers are optimally used when constructed adjacent to traffic. This application can be best visualized for a two span bridge with a pier located between median barriers. The use of precast piers minimizes traffic disruptions and construction work near traffic.

7.1.4.1.2 Application

Precast concrete piers have successfully been used on past projects. However, these projects did not allow the use of cast-in-place concrete piers which is currently not practical for most projects. An approach that allows for either cast-in-place or precast construction (or a combination thereof) after the contract has been awarded provides contractors greater flexibility to meet schedule demands, provides a safer work environment, and has the potential to reduce costs.

Optional precast concrete pier elements are currently being used on the I-39/90 Project. To aid in the continued development of precast piers, several bridges on the I-39/90 Project required the use of precast pier elements. These mandatory locations will follow the optional precast pier requirements, but prohibit cast-in-place construction. The remaining I-39/90 Project bridges, unless provided an exception, are being delivered as traditional cast-in-place piers with a noted allowance for the contractor to select a precast option. The precast option provides the Project Team and contractors with more flexibility while requiring minimal coordination with designers and the Bureau of Structures.

WisDOT policy item:

At this time, evaluation and plan preparations for accommodating a noted allowance for a precast pier option as indicated in this section is only required for I-39/90 Project bridges. All other locations statewide may consider providing a noted allowance for a precast option. Contact the Bureau of Structures Development Section for further guidance.

In some cases precast piers may not be suitable for a particular bridge location and there are specific limitations that can cause concern. The designer shall investigate the potential viability of using of precast pier elements for any proposed bridge. The designer should be aware of the common criteria for use and the limitations of the pier system. Some specific limitations for the optional precast pier element usage are the following:

- Piers shall be designed to allow either cast-in-place or precast concrete construction, but with only cast-in-place detailed on the plans. Differences between construction methods shall be limited to pier column connections, beam seats details, and diaphragm details. If the pier configuration is not able to reasonably accommodate interchangeability between the two constructions optional piers may be exempt from the precast option.

- Multi-column piers (3x4 ft rectangular) grade separations over roadways only.

- Fixed piers supporting prestressed concrete girders only.

- Precast elements shall be limited to 90 kips.

- Deep foundations are recommended when multiple pier caps are used. Shallow foundations may be considered if differential settlement is not expected.

- Integral barriers or crashwalls are currently excluded from the precast option.

- Applications where the top of the footing may become submerged are prohibited.

An exception to the precast pier option may be given by the Bureau of Structures.

7.1.4.1.3 Design Considerations

Precast concrete piers shall be designed in conformance with the current *AASHTO LRFD*, in accordance with the WisDOT Bridge Manual, and as given in the Special Provisions.

The optional precast pier allowance shall be established as prescribed in the optional precast pier details and specifications to envelope design requirements between precast and cast-in-place concrete construction. Contract plans shall follow a traditional cast-in-place delivery, with the exception of a noted allowance for precast piers. If the contractor selects the precast option, the contractor shall submit shop drawings, sealed by a professional engineer, to the Bureau of Structures. The fabrication shall be in conformance with the current *AASHTO LRFD*, in accordance to the Bridge Manual, and as given in the Special Provisions. Payment for the precast option will be paid using the cast-in-place concrete bid items.

Refer to Chapter 7 Standards for additional design considerations.

7.1.4.2 Geosynthetic Reinforced Soil – Integrated Bridge Systems (GRS-IBS)

Geosynthetic Reinforced Soil-Integrated Bridge Systems (GRS-IBS) are composed of two main components: Geosynthetic Reinforced Soil (GRS) and Integrated Bridge Systems (IBS). GRS is an engineered fill of closely spaced alternating layers of compacted fill and geosynthetic reinforcement that eliminates the need for traditional concrete abutments. IBS is a quickly-built, potentially cost-effective method of bridge support that blends the roadway into the superstructure using GRS technology. This integration system creates a transition area that allows for uniform settlement between the bridge substructure and the roadway approach, alleviating the “bump at the bridge” problem caused from uneven settlement. The result of this system is a smoother bridge approach.

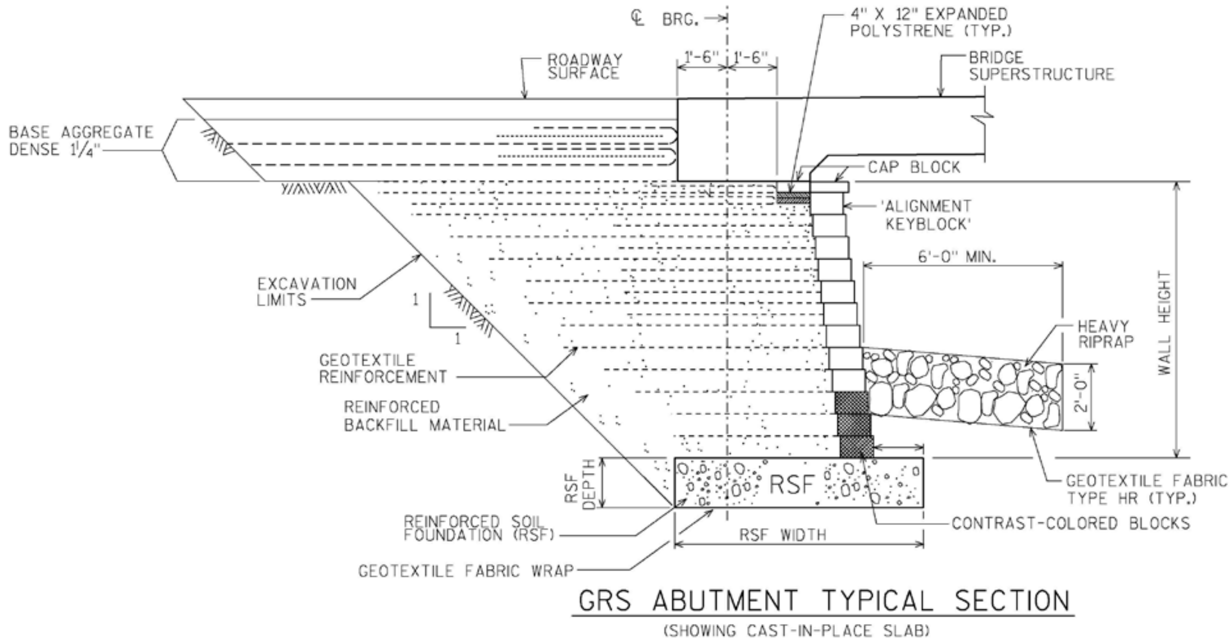

Figure 7.1-3
GRS-IBS Typical Cross Section

Figure 7.1-4
GRS-IBS Structure

Figure 7.1-5
GRS Abutment Layer During Construction

FHWA initially developed this accelerated construction technology, and the first bridge constructed in Wisconsin using the GRS-IBS technology was built in the spring of 2012. This structure (including structure numbers B-9-380, R-9-13, and R-9-14) is located on State Highway 40 in Chippewa County. This structure utilized a single-span cast-in-place concrete slab, which is the first of its kind in the nation. This structure will be closely monitored for two years to assess its performance.

This technology has several advantages over traditional bridge construction methods. A summary of the benefits of using GRS-IBS technology include the following:

1. Reduced Construction Time: Due to the simplicity of the design, low number of components, and only requiring common construction equipment to construct, the abutments can be rapidly built.

2. Potential Reduced Construction Costs: Compared to typical bridge construction in Wisconsin, GRS-IBS abutments can achieve significant cost savings. Nationwide, the potential cost savings is reported to be between 25 to 60% over traditional methods. The savings comes largely from the reduced number of construction steps, readily available and economical materials, and the need of only basic tools and equipment for construction.

3. Lower Weather Dependency: GRS-IBS abutments utilize only precast modular concrete facing blocks, open-graded backfill, and geotextile reinforcement in the basic design. The abutments can be constructed in poor weather conditions, unlike cast-in-place concrete, reducing construction delays.

4. Flexible Design: The abutment designs are simplistic and can be easily field-modified where needed to accommodate a variety of field conditions.

5. Potential Reduced Maintenance Cost: Since there are fewer parts to GRS-IBS abutments, overall maintenance is reduced. In addition, when repairs are needed, the materials are typically readily available and the work can be completed by maintenance staff or a variety of contractors.

6. Simpler Construction: The basic nature of the design demands less specialized construction equipment and the materials are usually readily available. Contractor capability and capacity demands are also reduced, allowing smaller and more diverse contractors to bid and complete the work.

7. Less Dependent on Quality Control: GRS-IBS systems are simple and basic in both their design and construction. Lack of technically challenging components and construction methods results in higher overall quality, reducing the probability of quality control related problems.

8. Minimized Differential Settlement: The GRS-IBS system is designed to integrate the structure with the approach pavement. Even though settlements can accumulate, differential settlement between the superstructure and the transition pavement is small. This can substantially reduce the common “bump at the bridge” that can be felt when traveling over traditional bridge transitions.

For more information, see Section 7.3, WisDOT Standard Details 7.01 and 7.02, and the Department’s specification.

7.1.4.2.1 Design Standards

GRS Abutments shall be designed in conformance with the current *AASHTO Load and Resistance Factor Design Specifications* (AASHTO LRFD) and in accordance with the WisDOT Bridge Manual.

7.1.4.2.2 Application

In some cases GRS-IBS abutments may not be suitable for a particular bridge location and there are specific limitations that can cause concern. As with any preliminary bridge planning, the site should be thoroughly investigated for adequacy. The designer shall investigate the potential viability of using of GRS-IBS for any proposed bridge. The designer should be aware of the common criteria for use and the limitations of GRS-IBS systems. Some of the common criteria for usage of GRS-IBS are the following:

1. Scour potential at the abutment locations has been evaluated and is within acceptable limits

2. Water velocities are less than 5 ft/s

3. Adequate freeboard is provided (See Bridge Manual Chapter 8.3.1.5)

4. Soil conditions permit shallow foundations.

5. Low-volume roadways

6. Single span structure with a span length less than 90 feet

7. Abutment wall height less than 22 feet (measured at the maximum wall height, from the top of the RSF to the top of the wall)

8. Wingwalls are parallel to roadway

9. Maximum skew angle of 15°

10. Short and long term settlements are tolerable

11. Differential settlement along the length of the abutment is tolerable to avoid twisting of the superstructure

12. Suitable construction materials available

7.1.4.2.3 Design Considerations

7.1.4.2.3.1 Hydraulics

Similar to any bridge spanning a waterway, the hydraulic conditions must be evaluated. The integrity of this system is very susceptible to scouring and undercutting of the Reinforced Soil Foundation (RSF) which could lead to further erosion and movement of the backfill in the GRS mass, causing settlement and possible structural failure.

WisDOT policy item:

The use of GRS-IBS is subject to prior-approval by the Bureau of Structures for hydraulic design. Evaluation of scour vulnerability will include assessment of long-term aggradation and degradation, potential for lateral migration of the stream, and calculation of contraction scour and abutment scour. The conservative nature of abutment scour calculations is acknowledged. Placement of adequately designed permanent scour countermeasures will be required to resist calculated scour.

In some cases of bridge replacement, the new GRS-IBS abutments can be constructed behind old abutments which can be left partially in place to promote scour protection for the RSF and GRS mass. Rip-rap, gabion mattresses and other traditional permanent counter measures can also be used.

To help bridge inspectors with scour detection, the lower rows of facing block below proposed grade should have an accent color (typically red, either integral or stained color treatment) that will become visible if scour is occurring. The accented colors provide a visual cue to inspectors that movement of soils has occurred. The top of the contrast-colored blocks shall be placed 2-3 block courses below the top of riprap elevation.

7.1.4.2.3.2 Reinforced Soil Foundation (RSF) and Reinforced Soil Mass

In the GRS-IBS system, bridge seat loads (including dead loads, live loads, etc.) and the weight of the GRS mass and facing blocks comprise the vertical loads that are carried by the RSF and ultimately transmitted to the soil. The vertical bridge seat loads are transferred to the RSF via the GRS mass. The facing blocks only carry their self-weight. Horizontal earth pressure forces are resisted by the GRS mass and little horizontal forces are carried by the facing blocks.

As with any bridge design, proper subsurface exploration should be conducted to ascertain the soil types and layer thicknesses in the vicinity of the proposed site. Laboratory testing may also be necessary to help determine the soil properties and provide the magnitude and time rate of total and differential settlements that may occur.

The external stability of the RSF and reinforced soil mass should be checked for failure against sliding, bearing capacity, and global stability. Due to the behavior of the reinforcement within the soil mass, overturning is an unlikely failure mode, but needs to be checked. The internal stability of the GRS mass should also be checked for bearing capacity, deformations, and the required reinforcement strength. FHWA (1) has provided general guidelines for GRS-IBS ultimate bearing capacities and the predicted deformations when using the prescribed material properties (geotextile, backfill, etc.) and geometry (layer spacings, wall height, etc.). In addition, anticipated settlements should be included when designing for vertical clearance. Under the conditions recommended by FHWA (1), creep in the geotextile reinforcement is typically negligible since the sustained stresses are redistributed and relatively low and reduction factors for creep are not required. Creep testing and evaluation should be conducted when the loading conditions and backfill and reinforcement conditions prescribed by FHWA (1) are exceeded.

The wall facing is composed of precast modular concrete blocks, which have a height of 8-inches. These types of blocks are readily available and need to conform to the same physical and chemical requirements as WisDOT MSE Wall Modular Blocks.

Special consideration should be given to the degree of batter of the various facing block systems. The amount of batter integrated into the wall systems can vary between manufacturers. Batter that is greater than expected will result in a decreased width between abutments when the span distance is held constant. The designer should be familiar with typical batter ranges for suppliers, and plan for variations in batter.

The wall facing blocks only support their self-weight and are held in place by the friction generated from their self-weight, the mechanical block interlocks, and the geotextile reinforcing fabric placed between each block layer. The upper layers of block will be less stable than the lower layers and they should be bonded in accordance with the specifications. This prevents movement of the blocks from expansion and contraction, freeze-thaw forces, settlement forces and vandalism.

The backfill should be an open graded material with an assumed internal angle of friction of 38 degrees. Generally this will limit the material to a crushed aggregate product. The RSF and integrated approach should generally use a wrapped dense graded aggregate.

The RSF and GRS mass should utilize a biaxial woven geotextile reinforcement fabric from the same manufacturer and of the same type and strength. Using biaxial geotextiles reduces the possibility of construction placement errors.

7.1.4.2.3.3 Superstructure

Typically, the bridge superstructure is placed directly on the reinforced soil abutment. Prestressed girders are often placed on top of the GRS substructure, followed by a traditional cast-in-place deck or precast deck panels. Other methods include the use of a cast-in-place concrete slab capable of spanning between the abutments or precast box girders. Both of these superstructure alternatives should be placed directly on the GRS abutment. The bearing area should contain additional geotextile reinforcement layers, which ensures that the superstructure bears on the GRS mass and not the facing blocks. The clear space between the facing block and the superstructure should be a minimum of 3-inches or 2 percent of the wall height, whichever is greater.

If steel or concrete I-girders are used, a precast or cast-in-place beam seat should be used to help distribute the girder reactions to the GRS abutment. Since there is open space between I-girders, the beam seat can be used to support a backwall between the girders to retain the soil behind the girder ends.

7.1.4.2.3.4 Approach Integration

The approach construction that ties the roadway to the superstructure is essential for minimizing approach settlement and minimizing the bump at each end of the bridge. With a GRS abutment, this is accomplished by compacting and reinforcing the approach fill in wrapped geotextile layers and blending the integration zone with the approach pavement structure.

The integrated approach is constructed in a similar manner as the GRS mass, using layers of geotextile reinforcement and aggregate backfill. However, the integrated approach uses thinner layers until approximately 2 inches from the bottom of the pavement structure. The lift thicknesses should not exceed 6-inches and should be adjusted to accommodate the beam depths.

7.1.4.2.3.5 Design Details

Many of the typical detailing requirements for traditional bridges are still required on GRS-IBS bridges such as railings, parapets, guardrail end treatments, and drainage. Steel posts should be used for guardrail systems within the GRS and integrated approach areas, which can more easily penetrate the layers of geotextile than timber posts.

Penetrations and disturbances through the geotextile layers should be kept to a minimum and only used when absolutely necessary. Planning the locations of utilities and future utilities should be considered to avoid disturbing these layers. If utilities must be installed through a GRS-IBS abutment, all affected layers of geotextile should be overlapped/spliced according to the manufacturer’s recommendations.

The backfill used for GRS-IBS is usually comprised of free draining, open graded material. The designer should give consideration to providing additional drainage if warranted. Surface drainage should be directed away from the wall face and the reinforced soil mass.

7.1.4.3 Lateral Sliding

Bridge placement using lateral sliding is another type of ABC where the entire superstructure is constructed in a temporary location and is moved into place over a night or weekend. This method is typically used for bridge replacement of a primary roadway where the new superstructure is constructed on temporary supports adjacent and parallel to the bridge being replaced. Once the superstructure is fully constructed, the existing bridge structure is demolished, and the new bridge is moved transversely into place. In some instances, a more complicated method known as a bridge launch has been used, which involves longitudinally moving a bridge into place.

Figure 7.1-6
Lateral Sliding

Several different methods have been used to slide a bridge into place. One common method is to push the bridge using a hydraulic ram while the bridge slides on a smooth surface and Teflon coated elastomeric bearing pads. Other methods have also been used, such as using rollers instead of sliding pads, and winches in place of a hydraulic ram. The bridge can also be built on a temporary support frame equipped with rails and pushed or pulled into place along those rails. Many DOTs have successfully replaced bridges overnight using lateral sliding.

This ABC method is used to replace bridges that are part of a main transportation artery traversing a minor road, waterway, or other geographic feature. The limiting factor with using lateral slide is having sufficient right-of-way, and space adjacent to the existing bridge to construct the new superstructure.

7.1.4.4 ABC Using Self Propelled Modular Transporter (SPMT)

7.1.4.4.1 Introduction

SPMTs are remote-controlled, self-leveling (each axle has its own hydraulic cylinder), multi-axle platform vehicles capable of transporting several thousand tons of weight. SPMTs have the ability to move laterally, rotate 360° with carousel steering, and typically have a jack stroke of 18 to 24 inches. They have traditionally been used to move heavy equipment that is too large for standard trucks to carry. SPMTs have been used for bridge placement in Europe for more than 30 years. Over the past decade, the United States has implemented SPMTs for rapid bridge replacement following the FHWA's recommendation in 2004 to learn how other countries have used prefabricated bridge components to minimize traffic disruption, improve work zone safety, reduce environmental impact, improve constructability, enhance quality, and lower life-cycle costs. The benefits of ABC using SPMTs include the following:

1. Minimize traffic disruption: Building or replacing a bridge using traditional construction methods can require the bridge to be closed for months to years, with lane restrictions, crossovers, and traffic slowing for the duration of the closure. Using SPMTs, a bridge can be placed in a matter of hours, usually requiring only a single night or weekend of full road closure and traffic divergence.

2. Improve work zone safety: The bridge superstructure is constructed in an off-site location called a bridge staging area (BSA). This allows construction of the entire superstructure away from live traffic, which improves the safety of both the construction workers and the traveling public.

3. Improve constructability: The BSA typically offers better construction access than traditional construction by keeping workspaces away from live traffic, environmentally sensitive areas, and over existing roadways.

4. Enhance quality: Bridge construction takes place off-site at the BSA where conditions can be more easily controlled, resulting in a better product. There is an opportunity to provide optimal concrete cure time in the BSA because the roadway in the temporary location does not have traffic pressures to open early.

5. Lower life-cycle costs: Because the quality of the bridge is increased, the overall durability and life of the bridge is also increased. This reduces the life-cycle cost of the structure.

6. Provide opportunities to include other ABC technologies: Multiple ABC technologies can be used on the same project, for example, a project could utilize prefabricated bridge elements, and also be moved into place using SPMTs.

7. Reduce environmental impacts: SPMT bridge moves have significantly shorter on-site construction durations than traditional construction, which is particularly

advantageous for areas that are environmentally sensitive. These areas may restrict on-site construction durations due to noise, light, or night work.

Figure 7.1-7
Self Propelled Modular Transporters Moving a Bridge

When replacing a bridge using SPMTs the new superstructure is built on temporary supports off-site in a designated BSA near the bridge site. Once the new superstructure is constructed, the existing structure can be removed quickly with SPMTs or can be demolished in conventional time frames, depending on the project-specific needs. Once the existing structure is removed, the new superstructure is moved from the staging area to the final location using two or more lines of SPMT units. The SPMTs lift the superstructure off of the temporary abutments and transport it to the permanent substructure. The placement of a bridge superstructure using SPMTs often requires only one night of full road closure, and many bridges in the United States have been placed successfully in a matter of hours.

When using SPMTs for bridge replacement a new substructure may be constructed, or the existing substructure may be reused. If the existing substructure is in good condition and meets current design requirements, it may be reused, or it may be rehabilitated. When constructing a new substructure, the new abutments are often built below the superstructure in front of the existing abutments, so the construction can advance before deconstruction of the existing structure begins. Because the superstructure is constructed in the BSA, the new superstructure can be constructed at the same time as the substructure.

SPMTs are typically used to replace bridges that carry or span major roadways. Time limitations or impacts to traffic govern the need for a quick replacement. Locating an off-site BSA to build the superstructure is a critical component for using SPMTs. There needs to be a clearly defined travel path (TP) between the staging area and the final bridge location that can support the SPMT movements (vertical clearances, horizontal clearances, turning radii, soil conditions, utility conflicts, etc.). See sections 7.1.4.4.6.1 and 7.1.4.4.6.2 for additional discussion of the BSA and TP.

SPMTs can also be used to place a bridge over a waterway. In this case, the bridge superstructure is constructed offsite, and then SPMTs transport the superstructure from the BSA onto a barge which travels the waterway to the final bridge site.

To date, mostly single-span bridges or individual spans of multi-span bridges with lengths ranging from approximately 100 to 200 feet have been moved with SPMTs. There have been a few two-span bridge moves with SPMTs in the United States. The most common structures that have been moved successfully are prestressed I-girder or steel plate girder bridges.

The following sections discuss key items for bridge placement using SPMT in the State of Wisconsin. For additional information on the use of SPMTs for the movement of bridges consult FHWA's *Manual on Use of Self Propelled Modular Transporters to Remove and Replace Bridges*, and UDOT's *SPMT Manual*. Contact the WisDOT Bureau of Structures Design Section as an additional resource.

7.1.4.4.2 Application

For guidance on whether SPMT bridge placement or another ABC technology should be used for a project, first refer to the WisDOT ABC decision making guidance spreadsheet and flowchart in Section 7.2. Some of the common criteria that govern the use of SPMTs are the following:

1. There is a need to minimize the out-of-service window for the roadway(s) on or under the structure
2. There is a major railroad track on or under the bridge
3. There is a major navigation channel under the bridge
4. The bridge is an emergency replacement

5. The road on or under the bridge has a high ADT and/or ADTT

6. There are no good alternatives for staged construction or detours

7. There is a sensitive environmental issue

Along with the use of this technology, the specifications need to include incentives and disincentives to employ for the project.

7.1.4.4.3 Special Provision

When writing a special provision for a project using SPMTs, consider the following items that may need to be included in the special provision text:

1. Drainage – Define areas (bridge site, BSA, TP, etc.) where drainage needs to be maintained throughout construction and indicate areas where temporary culvert pipes will be required. In the special provision text, clearly indicate if the temporary culvert pipes are to be included with the "SPMT Bridge Construction B-XX-XXX".

2. Temporary Concrete Barrier – define areas where temporary concrete barrier is required. Clearly indicate which barriers (temporary or permanent) are paid for with the roadway bid items, and which barriers are paid for with the item "SPMT Bridge Construction B-XX-XXX".

3. Bearing Pads – Indicate if bearing pads need to be adhered to the bottoms of girders prior to the bridge move or if temporary bearing pads are required on the temporary supports. Clearly indicate how the bearing pads are to be paid.

7.1.4.4.4 Roles and Responsibilities

The following sections outline the roles and responsibilities for the parties involved in the project using the design-bid-build delivery method. These roles apply if WisDOT specifies that the bridge will be placed using SPMTs. If SPMT use is not a stated requirement for the project, the Contractor may have the option to use them as long as the project specifications are met. If this occurs, the contractor would assume the responsibilities for certain items in Table 7.1-2 as described in 7.1.4.4.4.3.

Category	Responsibility Description	Responsible Party
Scoping	Decision to Use SPMTs	WisDOT Region & BOS
	Bridge Type Selection	Designer
	Provide Resources to Design Team	WisDOT BOS
Superstructure	Superstructure Design	Designer
Pick Points	Location and Tolerances	Designer
	Analyze Bridge for Effects from Lifting and Travel	Designer
Deflections	Set Stress, Deflection, and Twist Limits	WisDOT & Designer
	Monitoring Plan (Specifications)	Designer
	Monitoring Plan (Execution)	Contractor
BSA and TP	Location of BSA	Designer
	Geometry of TP	Designer
Utilities	Utility Agreements	WisDOT
	Mitigation Concepts	Designer
	Mitigation Execution	Contractor
Site Conditions	Structural Analysis of Bridge Along TP	Designer
	Set Allowable Stress Limits on BSA and TP	Designer
	Mitigation of Affected Areas at BSA and TP	Contractor
	Protection of Structure Along TP	Contractor
Heavy Lifter Equipment	SPMT	Contractor
	Heavy Lifter Equipment to Raise Bridge	Contractor
	Contingency Plan For Equipment Failure	Contractor
Support Structures	Permanent Substructure Design	Designer
	Temporary Support Design	Contractor

Table 7.1-2
SPMT Roles and Responsibilities

7.1.4.4.4.1 WisDOT

The WisDOT Region and the Bureau of Structures shall make the final decision to use SPMTs on a project, considering user costs. WisDOT either specifies to the designer that SPMTs will be used for the project, or they allow the contractor to propose an ABC method. If the latter is chosen, the project parameters, specification, schedule, and proposal should be defined in a way that ensures the requirements are met if the contractor decides that an SPMT move is the best solution.

7.1.4.4.4.2 Designer

The Designer includes any traffic, structural, or geotechnical engineers engaged by WisDOT in the design of the project. Final drawings and calculations should be stamped by a Professional Engineer licensed in the State of Wisconsin. The permanent substructure and superstructure should be designed in accordance with AASHTO LRFD Specifications and WisDOT Bridge Manual requirements. The superstructure should be designed to withstand induced forces from lifting off of temporary supports, transportation along TP, and lowering onto permanent bearings.

The Designer determines the feasibility of a BSA and TP, considering the following items at a minimum: geotechnical concerns, conflicting utilities, real estate and conflicting obstacles. The Designer also specifies the monitoring plan and maximum bearing pressure along travel path.

The Designer should deliver a project that can accommodate travel conditions during transportation of the structure on the SPMT units. Braking forces while the bridge is on the SPMTs shall be accounted for. Consider placing diaphragms at the pick points for additional lateral support.

7.1.4.4.4.3 Contractor

The Contractor may include the General Contractor, Heavy Lifter or SPMT Contractor, any bridge specialty engineers, and/or any other subcontractor employed by the General Contractor for the construction of the project.

The Contractor is responsible for:

1. The design of all temporary structures.
2. The construction of all structures, permanent or otherwise.
3. The design of the support system between the SPMT units and the bridge at final position.
4. The redesign and changes to plans to adjust for constructability issues based on the transport system chosen.
5. The design of the blocking or structure that supports the bridge during transport.
6. The safe transport of the bridge from the BSA to the final bridge location, ensuring that no maximum stresses or deflections are exceeded.

The Contractor is required to:

1. Provide all required plans, calculations, etc. in accordance with the specifications.

2. Identify, design and implement any required ground improvements in the BSA and TP.

3. Provide a contingency plan in the case of equipment malfunction or failure.

If the Contractor requests and is granted departmental approval to use SPMTs on a project that has not been designed for SPMT use, the following responsibilities (Refer to Table 7.1-2) that others are typically responsible for would be assumed by the Contractor:

1. Utilities – Mitigation Concepts

2. Site Conditions – Structural Analysis of Bridge Along TP

3. Site Conditions – Set allowable stress limits on BSA and TP

4. All Items under the category of Pick Points, Deflections (analysis), BSA and TP

5. Acquiring real estate

7.1.4.4.5 Temporary Supports

Temporary supports include temporary shoring and abutments that support the superstructure in the BSA and on the SPMTs during transport. The contractor is responsible for the design and construction of temporary supports. Temporary structures should be designed using *AASHTO Guide Design Specifications for Bridge Temporary Works*.

Design the temporary supports in the BSA to withstand a minimum lateral load equal to 10% of the superstructure dead load. Other lateral loads, such as wind, need not be included with this loading scenario.

These structures should provide bearing support conditions similar to the permanent bearings. The bridge superstructure is typically constructed in the temporary location with the same vertical clearance under the structure as the permanent location. The bridge may be constructed at a lower elevation for ease of construction; however this requires jacking the superstructure up to the correct elevation prior to transport.

SPMT blocking is the temporary support during transport that supports the superstructure at the pick point and connects to the SPMT units. Design SPMT blocking to withstand the forces induced during transport such as braking, turning, elevation changes, and wind loads.

7.1.4.4.6 Design Considerations

7.1.4.4.6.1 Bridge Staging Area

The BSA is the temporary location where the bridge superstructure will be constructed. The BSA is an area within the right of way, an offsite location, or an area acquired by the contractor. If an existing bridge is being removed using SPMTs, the BSA should provide adequate space for the superstructure to be removed. For projects with multiple bridges or one bridge with multiple simple spans, one or more bridges may occupy a single BSA. Figure 7.1-8 shows an example BSA that accommodated several structures.

Figure 7.1-8
Example Bridge Staging Area (BSA)

The BSA soil must have enough capacity to support the SPMTs carrying the superstructure. This requires a geotechnical investigation of the soils with possible additional measures such as ground improvements, soft soil mitigation, and utility protection. The contractor may need to address the bearing capacity of the soil in different manners based on the particular SPMT equipment that is selected. The BSA must be clear of all obstacles during bridge construction.

The designer specifies the maximum soil pressure in the BSA and TP based on the actual weight of the structure, anticipated SPMT weight, and temporary blocking. SPMT and temporary blocking weights need to be assumed. The design shall include a 5% dead load increase to cover miscellaneous loads (concrete tolerances, miscellaneous items, equipment during the move, etc.).

7.1.4.4.6.2 Travel Path

The TP is the path that the SPMTs use to transport the bridge(s) from the BSA to the final bridge location. The TP has similar requirements as the BSA. A geotechnical investigation is required to determine the need for ground improvements, soft soil mitigation, and utility protection. Steel plates, spreader beams, temporary pavement, and soft soil replacement are different methods used to help distribute the load and control settlement over these sensitive areas. Even a small area of soft soil can be detrimental during a superstructure transport. If the soil collapses under an SPMT tire, it can be extremely difficult to continue the bridge transport.

SPMT units are capable of traveling on uneven surfaces, however, it is preferred to keep the surface of the TP as level as possible with gradual elevation changes to minimize deflection and twist in the superstructure. Contact the WisDOT Bureau of Structures Design Section for approval of an uneven TP surface.

7.1.4.4.6.3 Allowable Stresses

During the process of lifting, transporting, and placing a bridge using SPMTs, the superstructure will undergo stresses different than those induced with traditional cast in place bridge construction. These stresses include stress reversals as described in 7.1.4.4.6.4. For calculation of the stresses in the superstructure when supported on the SPMTs, an impact factor of 1.15 applied to the dead load shall be used.

The Designer calculates the allowable stresses in the deck and in the girders. The bridge should be designed so that the reinforcement in the deck and parapet will not yield during transport of the bridge.

7.1.4.4.6.4 Pick Points

Pick points are the bearing locations where the superstructure is lifted off the temporary supports by the SPMTs and transported to the permanent location. Pick points should be located within 20% of the span length from the ends of the superstructure. This minimizes the cantilevered portion and negative forces induced on the superstructure. During the lifting of the superstructure off the temporary supports, the bridge undergoes a stress reversal. When the girders are placed and the deck is poured, the girders deflect under the wet concrete weight, inducing stresses in the girder. When the deck is cured, the stresses in the girders induced by the deck are locked in, and the superstructure is in a state of equilibrium. Changing the support locations causes a stress reversal in the superstructure, which must be considered in the design of the bridge.

Figure 7.1-9 illustrates the stress reversal that the superstructure undergoes when the bearing locations are changed. The easiest way to visualize this change is through the moment diagrams in the figure. The first diagram in the figure illustrates the moment on the superstructure due to dead loads with the support system at the ends similar to the final bearing system. The moment, M_a, is the moment at the pick point location. The second moment diagram shows the moments when the superstructure is supported at the pick points. Again, the moment, M_b, is the moment at the pick point location. The third diagram in the figure shows the two moments superimposed. The total stress that the superstructure sees at the pick point location, M_c, is from the two moments combined. Please note that this illustration is very simplified, and more in depth calculations and/or finite element modeling is required in order to calculate the actual stresses on the deck.

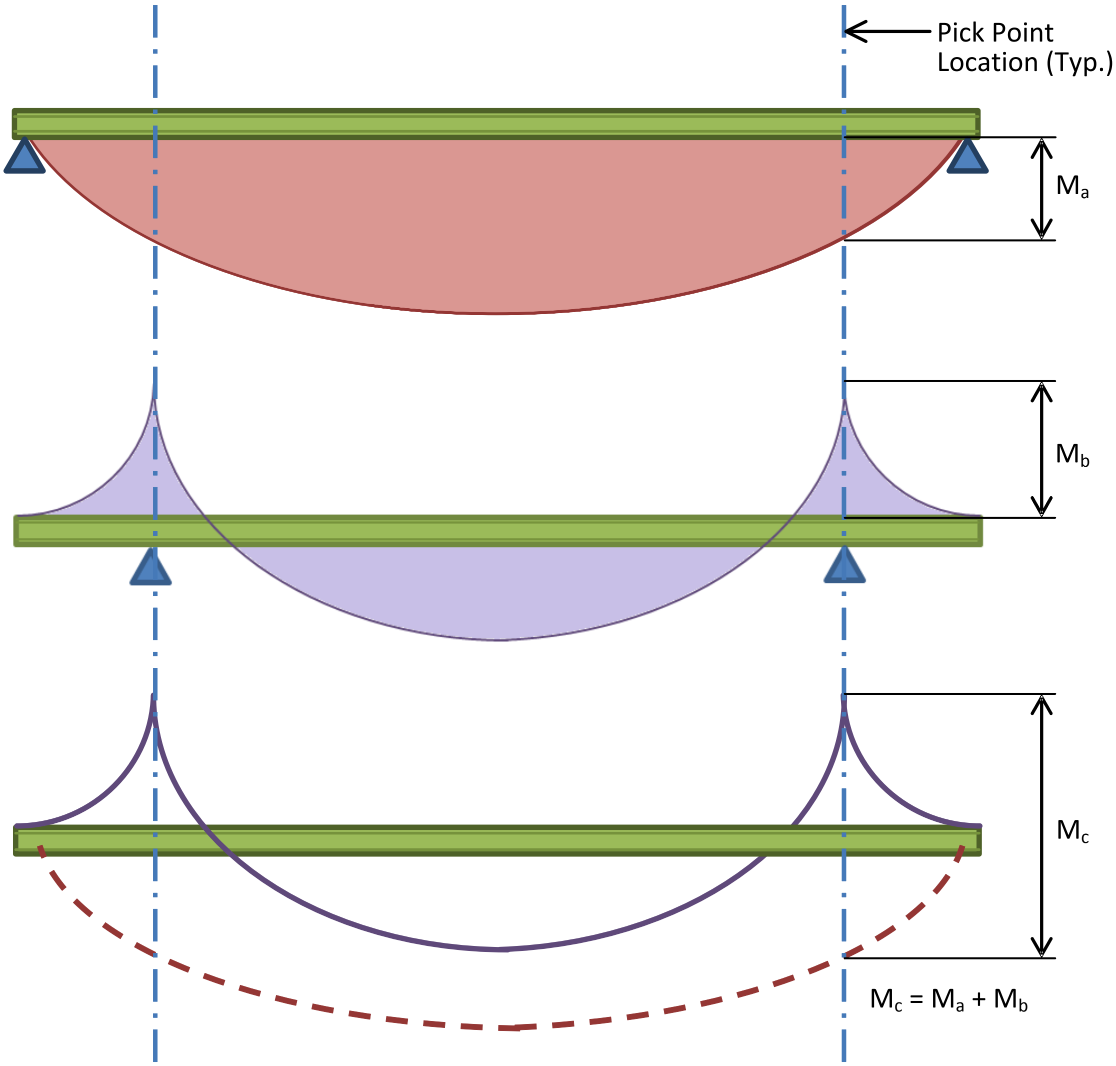

Figure 7.1-9
Support Change Moment Diagram (Illustrating Stress Reversal)

The construction sequence also complicates stress considerations. In the construction sequence, the girders are placed and the concrete is poured for the deck. The deck cures with essentially no stress, but the stress in the girders due to the deck pour is locked in when the girder and deck become composite. When the SPMTs engage the superstructure at the pick points, the girders go from positive bending at the pick points to negative bending. The deck at the pick point locations transitions from a state of zero bending (zero stress) to a state of negative bending. The stress calculations for the deck will be based on the composite moment of inertia.

The pick points must be located on the bridge in a manner to limit the tension in the deck. Clearly show pick points in the plans, and ensure that stresses induced from lifting and transporting the superstructure are within the allowable stresses shown in plans.

7.1.4.4.6.5 Deflection and Twist

During transport of the bridge from the BSA to its final position, the bridge will deflect and twist. Minor deflection and twist is to be expected during the movement of the bridge, but excessive deflections induce unwanted stresses in the deck that can cause cracking or other permanent damage to the superstructure. The bridge should be monitored during transport to keep the deflection and twist within specified limits. The specifications should outline the allowable deflections for the specific circumstances and structure(s). A critical point in the movement of the bridge is when the bridge is initially lifted off of the temporary supports. The stress reversal discussed in 7.1.4.4.6.4 will occur during this initial lift.

Warping and/or twisting of the bridge occurs when uneven bearing supports cause the slope of the bearing lines to be different from each other at each end of the span. Figure 7.1-10 shows an illustration of bridge warping. The blue solid square shows the as-constructed plane of the bridge. The red lines show the warped bridge plane and the dashed red lines represent the relative deflection from the as-constructed position.

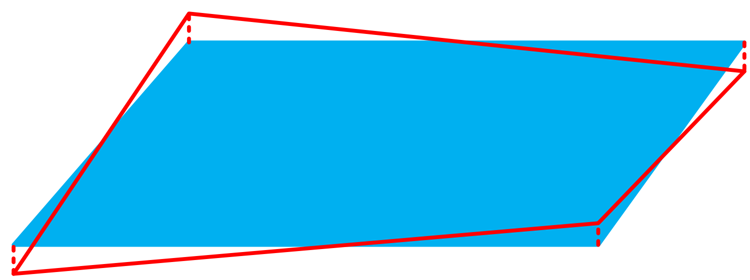

Figure 7.1-10
Bridge Warping Diagram

A monitoring plan should be developed by the Designer to monitor deflection and twist of the superstructure. Survey of critical points should be taken after construction of the superstructure and immediately after lifting it off of the temporary supports. A system should be established to monitor the relative deflections of each corner of the bridge during the transportation of the bridge. An example of bridge monitoring for deflection and twist can be found in UDOT's *Manual for the Moving of Utah Bridges Using Self Propelled Modular Transporters (SPMTs)*.

Accurate deflection calculations are very important when considering the SPMT unit jack stroke. For example, if the superstructure needs to be jacked 6 inches in order to lift the bridge off the temporary supports at the pick points, one quarter of the SPMT jack stroke would be used solely to lift the superstructure (assuming a typical jack stroke maximum of 24 inches).

Figure 7.1-11 illustrates how the deflection is accounted for in raising the superstructure off the temporary supports. Deflection, Δ_a, is the dead load deflection of the superstructure at the pick point location relative to the ends when the bridge is supported at the ends. Deflection, Δ_b, is the dead load deflection of the composite structure between the pick point location and the end support location when the bridge is supported at the pick point locations. Deflection, Δ_c, is the distance required to raise the structure off the temporary support.

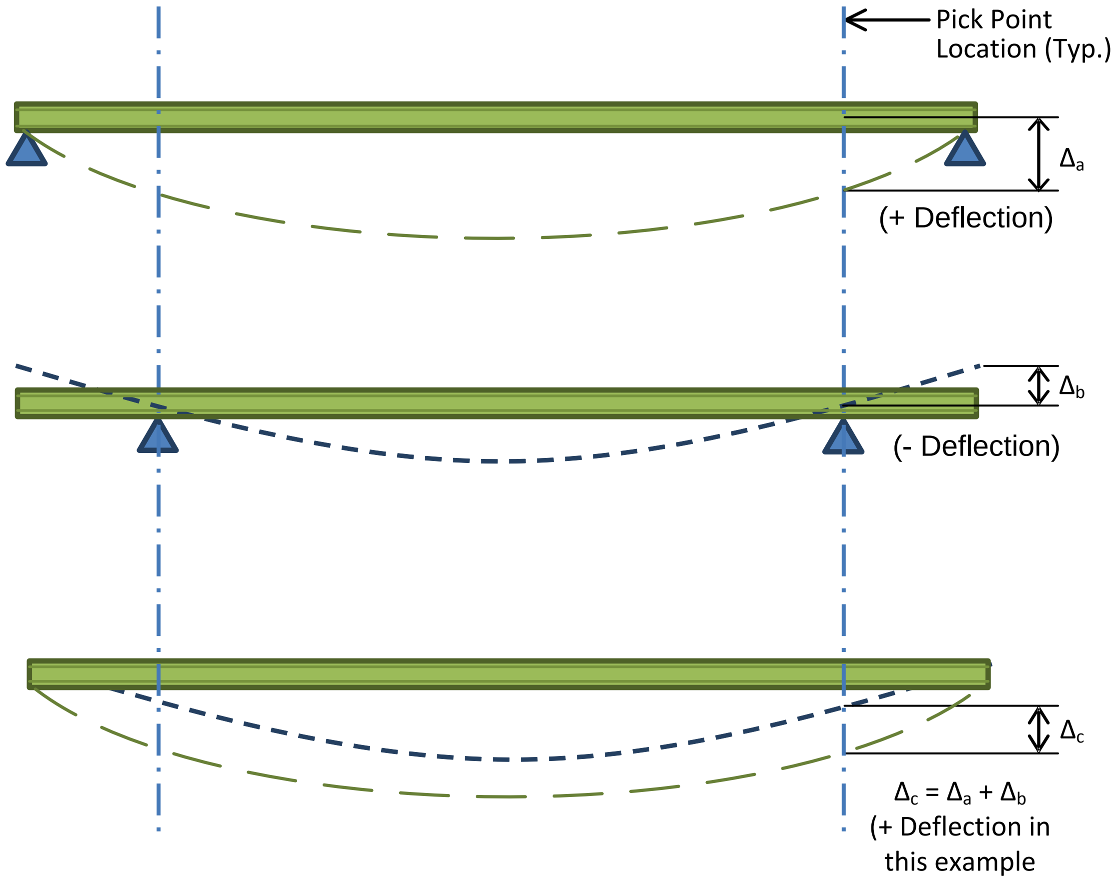

Figure 7.1-11
Support Change Deflection Diagram

Note: For this example, assume positive deflections are downward.

7.1.4.4.7 Structure Removal Using SPMT

When using SPMTs for bridge replacement, an alternative to onsite demolition of the existing bridge superstructure is removing the bridge using SPMTs. The existing superstructure can be removed and transported to the BSA where it is placed on temporary abutments until it can be demolished or salvaged. This method eliminates the need for protection of the underlying roadway and substructure elements.

All TP and BSA considerations, covered in 7.1.4.4.6.2 and 7.1.4.4.6.1 respectively, must be addressed for the movement of the existing superstructure. Follow guidelines in 7.1.4.4.5 for the design of temporary supports for existing superstructure.

7.1.5 Project Delivery Methods/Bidding Process

In addition to the accelerating technologies discussed in this chapter, the Every Day Counts initiative includes accelerated project delivery methods as a way to shorten the project duration. Traditionally, the Design-Bid-Build (DBB) method has been used for project delivery. This involves the design and construction to be completed by two different entities. Project schedules using the DBB method are elongated because the design and construction cannot be completed concurrently. The entire design process must be completed before the bidding process begins. Finally, after the bidding process is completed, the construction can begin.

Other state DOT's have used project delivery methods that can allow for more accelerated overall project delivery. These include Design/Build (D/B) and Construction Manager/General Contractor (CM/GC). The D/B process requires the designer-builder to assume responsibility for both the design and construction of the project. This method increases the risk for the design-builder, and reduces the risk for the owner. Project delivery time can be reduced, since the D/B process allows for the design and construction phases to overlap, unlike the DBB process. There is a specific type of D/B called Low Bid Design Build (LBDB) which has the same structure as the traditional D/B process, except that the lowest bidder wins the project (rather than having a quality component as with the traditional D/B process). Refer to the *Facilities Development Manual (FDM)* for further discussion on LBDB.

The CM/GC process is a hybrid of the DBB and D/B processes. In CM/GC, both the designer and the contractor have contracts with the owner, and the owner is part of the design team. In this process, a construction manager is selected, and is able to provide input regarding schedule, pricing, and phasing during the design phase. Around the 60% or 90% design completion, the owner and construction manager negotiate a "guaranteed maximum price" for the construction of the project based on the defined scope and schedule. CM/GC allows the owner to remain active in the design process, while the risk is still taken by the general contractor.

Generally, in Wisconsin, projects administered by the Department have been Design Bid Build with minimal use of the Low Bid Design Build method. Refer to the FDM Chapter 11-50-32 for additional discussion on Alternative Contracting (AC) methods.

WisDOT policy item:

Each state has different preferences and constraints to which project delivery method they use, and due to current legislation, CM/GC and traditional D/B are not viable options for the state of Wisconsin. To implement ABC using the DBB process, the contract should either specify to use the ABC method required by the owner, and/or provide opportunity for the contractor to propose ABC alternatives that meet contract requirements.

7.2 ABC Decision-Making Guidance

This section is intended to provide guidance on when to use ABC versus conventional construction. When ABC methods are appropriate, this section will also help determine which ABC method(s) are most practical for a particular project.

Figure 7.2-1 is a Decision Matrix that can be used to determine how applicable an ABC method is for a particular project. Each item in Figure 7.2-1 is described further in Table 7.2-1. Once a total score is obtained from the Decision Matrix, the score is used to enter the Decision Flowchart (Figure 7.2-2). After entering the Flowchart, the user could be directed to the question "Do the benefits of ABC outweigh any additional costs?" This question needs to be evaluated on a project-specific basis, using available project information and engineering judgment. This item is intended to force the user to step back, think about the project as a whole, and decide if an ABC method really makes sense with all the project-specific information considered. The remainder of the flow chart questions will help guide the user toward the ABC method(s) that are most appropriate for the project.

There is an acknowledged level of subjectivity in both the Decision Matrix and in the Flowchart. These tools are intended to provide general guidance, not to provide a specific answer for all projects. The tools present different types of considerations that should be taken into account to help guide the user in the right direction and are not intended to provide a "black and white" answer.

The flowchart item "Program Initiative" can encompass a variety of initiatives, including (but not limited to) research needs, public input, local initiatives, stakeholder requests, or structure showcases. These items should be considered on a project-specific basis.

The flowchart guides users towards specific ABC technologies. However, the user should also recognize the ability and opportunity to combine various ABC technologies. For example, the combination of PBES with GRS-IBS could be utilized.

For additional guidance or questions, contact the Bureau of Structures Development Section Chief.

% Weight	Category	Decision-Making Item	Possible Points	Points Allocated	Scoring Guidance
17%	Disruptions (on/under Bridge)	**Railroad on Bridge?**	8		0 No railroad track on bridge 4 Minor railroad track on bridge 8 Major railroad track on bridge
		Railroad under Bridge?	3		0 No railroad track under bridge 1 Minor railroad track under bridge 3 Major railroad track(s) under Bridge
		Over Navigation Channel that needs to remain open?	6		0 No navigation channel that needs to remain open 3 Minor navigation channel that needs to remain open 6 Major navigation channel that needs to remain open
8%	Urgency	**Emergency Replacement?**	8		0 Not emergency replacement 4 Emergency replacement on minor roadway 8 Emergency replacement on major roadway
23%	User Costs and Delays	**ADT and/or ADTT** (Combined Construction Year ADT on and under bridge)	6		0 No traffic impacts 1 ADT under 10,000 2 ADT 10,000 to 25,000 3 ADT 25,000 to 50,000 4 ADT 50,000 to 75,000 5 ADT 75,000 to 100,000 6 ADT 100,000+
		Required Lane Closures/Detours? (Length of Delay to Traveling Public)	6		0 Delay 0-5 minutes 1 Delay 5-15 minutes 2 Delay 15-25 minutes 3 Delay 25-35 minutes 4 Delay 35-45 minutes 5 Delay 45-55 minutes 6 Delay 55+ minutes
		Are only Short Term Closures Allowable?	5		0 Alternatives available for staged construction 3 Alternatives available for staged construction, but undesirable 5 No alternatives available for staged construction
		Impact to Economy (Local business access, impact to manufacturing etc.)	6		0 Minor or no impact to economy 3 Moderate impact to economy 6 Major impact to economy
14%	Construction Time	**Impacts Critical Path of the Total Project?**	6		0 Minor or no impact to critical path of the total project 3 Moderate impact to critical path of the total project 6 Major impact to critical path of the total project
		Restricted Construction Time (Environmental schedules, Economic Impact – e.g. local business access, Holiday schedules, special events, etc.)	8		0 No construction time restrictions 3 Minor construction time restrictions 6 Moderate construction time restrictions 8 Major construction time restrictions
5%	Environment	**Does ABC mitigate a critical environmental impact or sensitive environmental issue?**	5		0 ABC does not mitigate an environmental issue 2 ABC mitigates a minor environmental issue 3 ABC mitigates several minor environmental issues 4 ABC mitigates a major environmental issue 5 ABC mitigates several major environmental issues
3%	Cost	**Compare *Comprehensive* Construction Costs** (Compare conventional vs. prefabrication)	3		0 ABC costs are 25%+ higher than conventional costs 1 ABC costs are 1% to 25% higher than conventional costs 2 ABC costs are equal to conventional costs 3 ABC costs are lower than conventional costs
18%	Risk Management	**Does ABC allow management of a particular risk?**	6		0 - 6 Use judgment to determine if risks can be managed through ABC that aren't covered in other topics
		Safety (Worker Concerns)	6		0 Short duration impact with TMP Type 1 3 Normal duration impact with TMP Type 2 6 Extended duration impact with TMP Type 3-4
		Safety (Traveling Public Concerns)	6		0 Short duration impact with TMP Type 1 3 Normal duration impact with TMP Type 2 6 Extended duration impact with TMP Type 3-4
12%	Other	**Economy of Scale** (repetition of components in a bridge or bridges in a project) (Total spans = sum of all spans on all bridges on the project)	5		0 1 total span 1 2 total spans 2 3 total spans 3 4 total spans 4 5 total spans 5 6+ total spans
		Weather Limitations for conventional construction?	2		0 No weather limitations for conventional construction 1 Moderate limitations for conventional construction 2 Severe limitations for conventional construction
		Use of Typical Standard Details (Complexity)	5		0 No typical standard details will be used 3 Some typical standard details will be used 5 All typical standard details will be used
			Sum of Points:	0	(100 Possible Points)

Figure 7.2-1

ABC Decision-Making Matrix

7.2.1 Descriptions of Terms in ABC Decision-Making Matrix

The following text describes each item in the ABC Decision-Making Matrix (Figure 7.2-1). The points associated with the scoring guidance in the matrix and in the text below are simply *guidance*. Use engineering judgment and interpolate between the point ranges as necessary.

Decision-Making Item	Scoring Guidance Description
Railroad on Bridge?	This is a measure of how railroad traffic on the bridge will be affected by the project. If a major railroad line runs over the bridge that requires minimum closures and a shoo fly (a temporary railroad bridge bypass) cannot be used, provide a high score here. If a railroad line that is rarely used runs over the bridge, consider providing a mid-range or low score here. If there is no railroad on the bridge, assign a value of zero here.
Railroad under Bridge?	This is a measure of how railroad traffic under the bridge will be affected by the project. If a major railroad line runs under the bridge that would disrupt construction progress significantly, provide a high score here. If a railroad track runs under the structure, but it is used rarely enough that it will not disrupt construction progress significantly, provide a low score here. Consider if the railroad traffic is able to be suspended long enough to move a new bridge into place. If there is not a large enough window to move a new bridge into place, SPMT could be eliminated as an alternative for this project. For this case, PBES may be a more applicable alternative. If there is no railroad under the bridge, assign a value of zero here.
Over Navigation Channel that needs to remain open?	This is a measure of how a navigation channel under a bridge will be affected by the project. If a navigation channel is highly traveled and needs to remain open for shipments, provide a high score here. If a navigation channel is rarely traveled and there are not requirements for it to remain open at certain time periods, provide a low score here. If there is no navigation channel under the bridge, assign a value of zero here.
Emergency Replacement?	This is a measure of the urgency of the bridge replacement. A more urgent replacement supports the use of accelerated bridge construction methods, since demolition and construction can be progressing concurrently. Depending on the particular project, accelerated bridge construction methods can also allow multiple components of the bridge to be constructed concurrently. If the bridge replacement is extremely urgent and the bridge can be replaced quicker by using accelerated construction methods, provide a high score here.

ADT and/or ADTT (Construction Year)	This is a measure of the total amount of traffic crossing the bridge site. A higher ADT value at a site will help support the use of accelerated bridge construction methods. Use a construction year ADT value equal to the sum of the traffic on the structure and under the structure. For cases where there is a very high ADT on the bridge and very low or no ADT under the bridge, consider using a "slide" method (on rollers or Polytetrafluorethylene (PTFE)/Elastomeric pads) or SPMT's, which can be very cost effective ABC techniques for this situation. For structures with a higher-than-average percentage of truck traffic, consider providing a higher score than indicated solely by the ADT values in the table.
Required Lane Closures/Detours?	This is a measure of the delay time imposed on the traveling public. If conventional construction methods will provide significant delays to the traveling public, provide a high score here. If conventional construction methods will provide minimal delays to the traveling public, provide a low score here. Use the delay times provided in the table as guidance for scoring.
Are only Short Term Closures Allowable?	This is a measure of what other alternatives are available besides accelerated bridge construction. If staged construction is not an alternative at a particular site, the only alternative may be to completely shut down the bridge for an SPMT move, and therefore a high score should be provided here. If there is a good alternative available for staged construction that works at the site, a low score should be provided here.
Impact to Economy	This is a measure of the impact to the local businesses around the project location. Consider how the construction staging, road closures, etc. will impact local businesses (public access, employee access, etc.) A high impact to the economy equates to a high score here. A low impact to the economy equates to a low score here.
Impacts Critical Path of Total Project?	This is a measure of how the construction schedule of the structure impacts the construction schedule of the entire project. If the construction of the structure impacts the critical path of the entire project, and utilizing ABC methods provides shorter overall project duration, provide a high score here. If other project factors are more critical for the overall project schedule and utilizing ABC methods will not affect the overall project duration, provide a low score here.
Restricted Construction Time	This is a measure of how the construction schedule is impacted by environmental and community concerns or requirements. Items to consider are local business access windows, holiday schedules and traffic, special event traffic, etc. If there are significant restrictions on construction schedule, provide a high score here. If there are little to no restrictions on the construction schedule, provide a low score here.

Does ABC mitigate a critical environmental impact or sensitive environmental issue?	This is a measure of how using accelerated bridge construction methods can help mitigate impacts to the environment surrounding the project. Since accelerated methods allow a shorter on-site construction time, the impacts to the environment can be reduced. If the reduced on-site construction time provided by accelerated bridge construction methods mitigates a significant or critical environmental concern or issue, provide a high score here. If there are no environmental concerns that can be mitigated with accelerated construction methods, provide a low score here.
Compare Comprehensive Construction Costs	This is a measure of the complete comprehensive cost difference between conventional construction methods versus using an accelerated bridge construction method. Some costs will increase with the use of accelerated construction methods, such as the cost of the SPMT equipment and the learning curve that will be incorporated into using new technologies. However, some costs will decrease with the use of accelerated construction methods, such as the reduced cost for traffic control, equipment rentals, inspector wages, etc. Many of the reduced costs are a direct result of completing the project in less time. Use the cost comparisons in the table as guidance for scoring here.
Does ABC allow management of a particular risk?	This is an opportunity to add any project-specific items or unique issues that have risk associated with them that are not incorporated into another section in this text. Consider how ABC may or may not manage those particular risks.
Safety (Worker Concerns)	This is a measure of the relative safety of the construction workers between conventional construction methods and accelerated construction methods. The reduced on-site construction time from using accelerated bridge construction methods reduces the exposure time of workers in a construction zone, thus increasing safety. If a significant increase in safety can be seen by utilizing accelerated construction methods, provide a high score here. If utilizing accelerated construction methods does not provide additional safety, provide a low score here. Refer to the *Facilities Development Manual (FDM)* for definitions of TMP Types.
Safety (Traveling Public Concerns)	This is a measure of the relative safety of the traveling public between conventional construction methods and accelerated construction methods. The reduced on-site construction time from using accelerated bridge construction methods reduces the exposure time of the traveling public in a construction zone, thus increasing safety. If a significant increase in safety can be seen by utilizing accelerated construction methods, provide a high score here. If utilizing accelerated construction methods does not provide additional safety, provide a low score here. Refer to the *Facilities Development Manual (FDM)* for definitions of TMP Types.

Economy of Scale	This is a measure of how much repetition is used for elements on the project, which can help keep costs down. Repetition can be used on both substructure and superstructure elements. To measure the economy of scale, sum the total number of spans that will be constructed on the project. For example, if there are 2 bridges on the project that each have 2 spans, the total number of spans on the project is equal to 4. Use the notes in the table for scoring guidance here.
Weather Limitations for Conventional Construction?	This is a measure of the restrictions that the local weather causes for on-site construction progress. Accelerated bridge construction methods may allow a large portion of the construction to be done in a controlled facility, which helps reduce delays caused by inclement weather (rain, snow, etc.). Depending on the location and the season, faster construction progress could be obtained by minimizing the on-site construction time.
Use of Typical Standard Details (Complexity)	This is a measure of the efficiency that can be gained by using standard details that have already been developed and approved. If standard details are used, some errors in the field can be prevented. If new details are going to be created for a project, the contractors will be less familiar with the details and problems may arise during construction that were not considered in the design phase. Use the notes in the table for scoring guidance here.

Table 7.2-1
ABC Decision-Making Matrix Terms

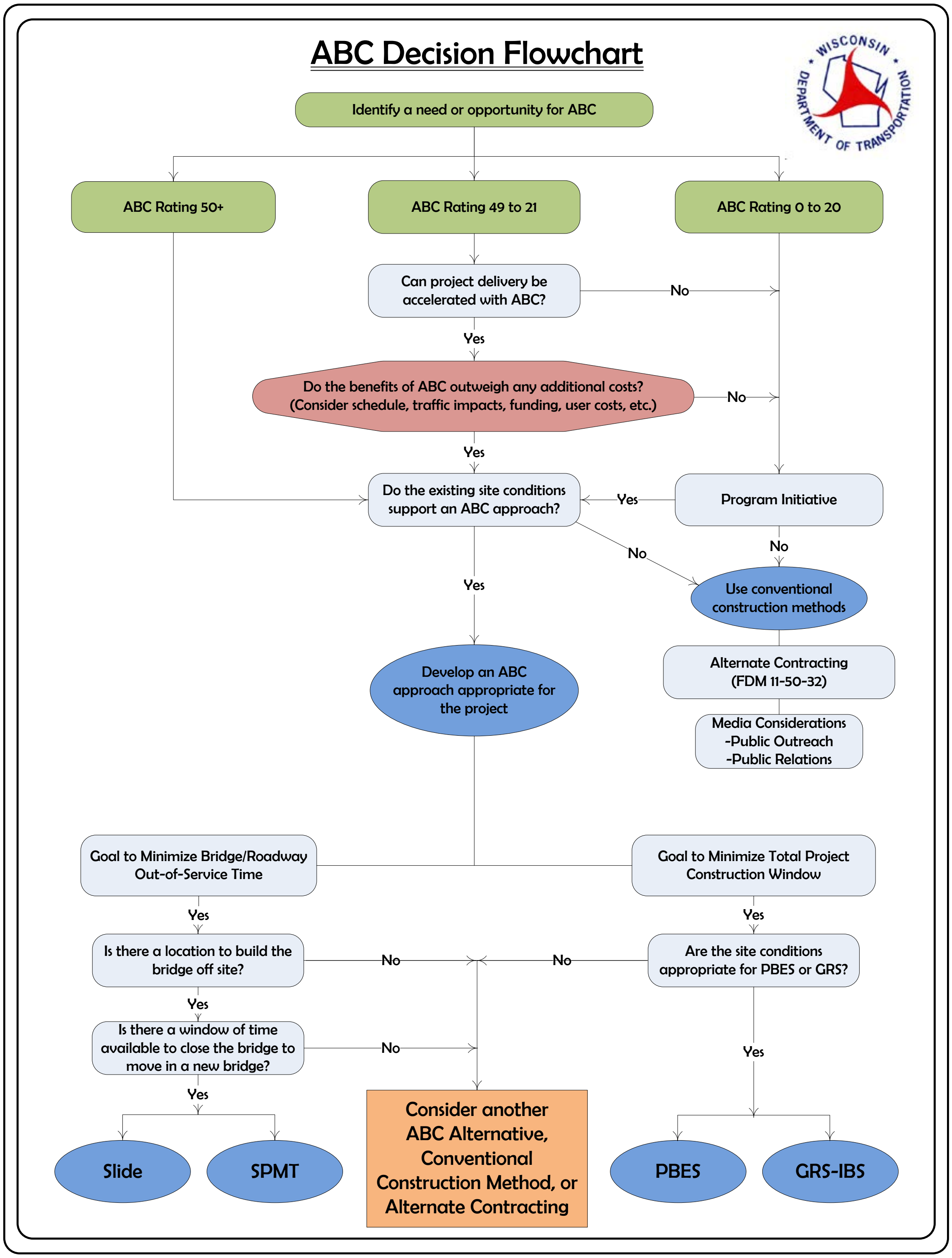

Figure 7.2-2
ABC Decision-Making Flowchart

7.3 References

1. Every Day Counts Initiative. Federal Highway Administration. 23 May. 2012. http://www.fhwa.dot.gov/everydaycounts/

2. Federal Highway Administration. Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide. U.S. Department of Transportation. McLean, VA: Turner-Fairbank Highway Research Center, 2011. FHWA-HRT-11-026

3. Federal Highway Administration. Geosynthetic Reinforced Soil Integrated Bridge System Synthesis Report. U.S. Department of Transportation. McLean, VA: Turner-Fairbank Highway Research Center, 2011. FHWA-HRT-11-027.

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9.1 General

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

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

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

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

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

9.4 Steel

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

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

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

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

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

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

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

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

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

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

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

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

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10.1 General

The purpose of the Geotechnical Investigation is to provide subsurface information for the plans and to develop recommendations for the construction of the structure at reasonable costs versus short and long term performance. The level of Geotechnical Investigation is a function of the type of the structure and the associated performance. For example, a box culvert under a low ADT roadway compared to a multi-span bridge on a major interstate would require a different level of Geotechnical Investigation. The challenge for the geotechnical engineer is to gather subsurface information that will allow for a reasonable assessment of the soil and rock properties compared to the cost of the investigation.

The geotechnical engineer and the structure engineer need to work collectively when evaluating the loads on the structures and the resistance of the soil and rock. The development of the geotechnical investigation and evaluation of the subsurface information requires a degree of engineering judgment. A guide for performing the Geotechnical Investigation is provided in WisDOT Geotechnical Bulletin No. 1, **LRFD [10.4]** and Geotechnical Engineering Circular #5 – Evaluation of Soil and Rock Properties (Sabatini, 2002).

The following structures will require a Geotechnical Investigation:

- Bridges
- Box Culverts
- Retaining Walls
- Non-Standard Sign Structures Foundations
- High Mast Lighting Foundations
- Noise Wall Foundations

10.2 Subsurface Exploration

The Geotechnical Engineering Unit (or geotechnical consultant) prepares the Site Investigation Report (SIR) and the Subsurface Exploration (SE) sheet. The SIR describes the subsurface investigation, laboratory testing, analyses, computations and recommendations for the structure. All data relative to the underground conditions which may affect the design of the proposed structure's foundation are reported. Further information describing this required investigation can be found in the Department's "Geotechnical Bulletin #1" document. The Subsurface Exploration sheet is a CADDS drawing that illustrates the soil boring locations and is a graphical representation of the driller's findings. This sheet is included in the structure plans. If the Department is not completing the geotechnical work on the project, the SIR and SE sheet(s) are the responsibility of the consultant.

The subsurface investigation is composed of two areas of investigation: the Surface Survey and the detailed Site Investigation.

Surface Surveys include studies of the site geology and air-photo review, and they can include geophysical methods of exploration. This work should include a review of any existing structure foundations and any existing geotechnical information. Surface Surveys provide valuable data indicating approximate soil conditions during the reconnaissance phase.

Based on the results of the Surface Survey information, the plans for a Detailed Site Investigation are made. The subsurface investigation needs to provide the following information:

- Depth, extent and thickness of each soil or rock stratum
- Soil texture, color, mottling and moisture content
- Rock type, color and condition
- In-situ field tests to determine soil and rock parameters
- Laboratory samples for determining soil or rock parameters
- Water levels, water loss during drilling, utilities and any other relevant information

The number and spacing of borings is controlled by the characteristics and sequence of subsurface strata and by the size and type of the proposed structure. Depending upon the timing of the Geotechnical Investigation the required information may not be available and the geotechnical engineer may have to develop a subsurface investigation plan based on the initial design. The Department understands that additional investigation may be required once the preliminary design is completed. The challenge for the Department and the consultant is to develop a geotechnical investigation budget without knowing the subsurface conditions that will be encountered. Existing subsurface information from previous work can help this situation, but the plans should be flexible to allow for some unforeseen subsurface conditions.

One particular subsurface condition is the presence of shallow rock. In some cases, borings should be made at a frequency of one per substructure unit to adequately define the subsurface conditions. However, with shallow rock two or more borings may be necessary to define the rock line below the foundation. Alternatively, where it is apparent the soil is uniform, fewer borings are needed. For example, a four span bridge with short (less than30 foot) spans at each end of a bridge may only require three borings versus the five borings (one per substructure).

Borings are typically advanced to a depth where the added stress due to the applied load is 10 percent of the existing stress due to overburden or extended beyond the expected pile penetration depths. Where rock is encountered, borings are advanced by diamond bit coring according to ASTM D2113 to determine rock quality according to ASTM D6032.

LRFD [Table 10.4.2-1] Minimum Number of Exploration Points and Depth of Exploration (modified after Sabatini et al., 2002) provides guidelines for an investigation of bridges (shallow foundations and deep foundations) and retaining walls. The following presents the typical subsurface investigation guidelines for the other structures:

- Box Culverts: The recommended spacing of the borings would be 1/every 200 feet of length of the box culvert with a minimum of two boring for a new box culvert. The borings should have 15 feet of continuous SPT samples below the base of the box culvert.

- Box Culvert Extensions: May require a boring depending upon the length of the extension and the available information from the existing box culvert. If a boring is recommended then it would follow the same procedures as for a new box culvert.

- Non-Standard Sign Structure Foundations: The recommended spacing would be one for each sign structure site. If the sign structure is a bridge with two foundations then one boring may still be adequate. The borings should have 20 feet of continuous SPT samples and a SPT sample at 25 feet and 30 feet below the ground surface at the sign structure site.

- High Mast Lighting Foundations: The recommended spacing would be one for each site. The borings should have 15 feet of continuous SPT samples and a SPT sample every 5 feet to a depth of 40 feet below the ground surface at the site.

- Noise Wall Foundations: The recommended spacing would be one for every 200 feet to 300 feet of wall. The borings should have 20 feet of continuous SPT samples below the ground surface.

The Department generally follows AASHTO laboratory testing procedures. Any or all of the following soil tests may be considered necessary or desirable at a given site:

In-situ (field) Tests

- Standard penetration
- Pocket penetrometer (cohesive soil)
- Vane shear (cohesive soil)
- Cone penetration (seldom used)
- Rock core recovery and Rock Quality Designation (RQD)

Laboratory Tests

- Moisture, density, consistency limits and unit weight
- Unconfined compression (cohesive soils and rock cores)
- Grain size analysis (water crossings) - This test is required for streambed sediments of multi-span structures over water to facilitate scour computations.
- One-dimensional consolidation (seldom used)
- Unconsolidated undrained triaxial compression (seldom used)
- Consolidated undrained triaxial compression with pore water pressure readings (seldom used)
- Corrosion Tests (pH, resistivity, sulfate, chloride and organic content)

One of the most widely used in-situ tests in the United States is the Standard Penetration Test (AASHTO T-206) as described in the *AASHTO Standard Specifications*. This test provides an indication of the relative density of cohesionless soils and, along with the pocket penetrometer readings, predicts the consistency and undrained shear strength of cohesive soils. Standard Penetration Tests (SPTs) generally consist of driving a 2-inch O.D. split barrel sampler into the ground with a 140-pound hammer falling over a height of 30 inches. The split-barrel sampler is driven in 6-inch increments for a total of 18-inches and the number of blows for each 6-inch increment is recorded. The field blow-count, SPT N-value, equals the number of blows that are required to drive the sampler the last 12-inches of penetration. Split-barrel samplers are typically driven with a conventional donut, safety or automatic-trip hammer. Hammer efficiencies, ER, are determined in accordance with ASTM D 4945. In lieu of a more detailed assessment, ER values of 45, 60 and 80 percent may be used to compute corrected blow counts, N_{60}, for conventional, safety and automatic-trip hammers, respectively, in accordance with **LRFD [10.4.6.2.4]**. Correlation between standard penetration values and the resulting soil bearing value approximations are available from many sources. Standard penetration values can be used by experienced Geotechnical Engineers to estimate pile shaft resistance values by also considering soil texture, moisture content, location of water table, depth below proposed footing and method of boring advance.

For example, DOT Geotechnical Engineers using DOT soil test information know that certain sand and clays in the northeastern part of Wisconsin have higher load-carrying capacities than tests indicate. This information is confirmed by comparing test pile data at the different sites to computed values. The increased capacities are realized by increasing the design point resistance and/or shaft resistance values in the Site Investigation Report.

Wisconsin currently uses most of the soil tests previously mentioned. The soil tests used for a given site are determined by the complexity of the site, size of the project and availability of funds for subsurface investigation. The scope and extent of the laboratory testing program should take into consideration available subsurface information obtained during the initial site reconnaissance and literature review, prior experience with similar subsurface conditions encountered in the project vicinity and potential risk to structure performance. Detailed information about how to develop a laboratory testing program and the type of tests required is presented in previous sited reference or refer to a soils textbook for a more detailed description of soil tests.

Laboratory tests of undisturbed samples provide a more accurate assessment of soil settlement and structural properties. Unconfined compression tests and other tests are employed to measure the undrained shear strength and to estimate pile shaft resistance in clay soils by assuming:

$$c = \frac{q_u}{2}$$

Where:

c	=	cohesion of soil
q_u	=	unconfined compression strength

It is worthy to note that pile shaft resistance is a function of multiple parameters, including but not limited to stress state, depth, soil type and foundation type.

In addition to the tests of subsurface materials, a geological and/or geophysical study may be conducted to give such geological aspects as petrology, rock structure, rock quality, stratigraphy, vegetation and erosion. This can include in-situ and laboratory testing of selected samples, as well as utilizing non-destructive geophysical techniques, such as seismic refraction, electromagnetic or ground penetrating radar (GPR)

Boring and testing data analysis, along with consideration of the geology and terrain, allow the geotechnical engineer to present the following in the bridge SIR:

- The preferred type of substructure foundation (i.e. shallow or deep).
- The factored bearing resistance for shallow foundations.

- The settlement for the shallow foundations.

- If piles are required, recommend the most suitable type and the support values (shaft resistance and point resistance) furnished by the different soil strata.

- A discussion of any geotechnical issues that may affect construction.

- The presence and affect of water, including discussion of dewatering impact and cut-slope impact under abutments.

When piles are recommended, suitable pile types, estimated length requirements, pile drivability and design loads are discussed. Adverse conditions existing at abutments due to approach fills being founded on compressible material are pointed out, and recommended solutions are proposed. Unfactored resistance values at various elevations are given for footing foundation supports. Problems associated with scour, tremie seals, cofferdams, settlement of structure or approach fill slopes and other conditions unique to a specific site are discussed as applicable.

10.3 Soil Classification

The total weight of the structure plus all of the forces imposed upon the structure is carried by the foundation soils. There are many ways to classify these soils for foundation purposes. An overall geological classification follows:

1. Bedrock - This is igneous rock such as granite; sedimentary rock such as limestone, sandstone and shale; and metamorphic rock such as quartzite or marble.

2. Glacial soils (Intermediate Geo Material- IGM) - This wide variety of soils includes granular outwash, hard tills, bouldery areas and almost any combination of soil that glaciers can create and are typically defined to have a SPT number greater than 50.

3. Alluvial soils - These are found in flood plains and deltas along creeks and rivers. In Wisconsin, these soils normally contain large amounts of sand and silt. They are highly stratified and generally loose. Pockets of clay are found in backwater areas.

4. Residual soils - These soils are formed as a product of weathering and invariably reflect the parent bedrock material. They may be sands, silts or clay.

5. Lacustrine soils - These soils are formed as sediment and are deposited in water environments. In Wisconsin, they tend to be clayey. One example of these soils is the red clay sediments around Lakes Superior and Michigan.

6. Gravel, cobbles and boulders - These are particles that have been dislodged from bedrock, then transported and rounded by abrasion. Some boulders may result from irregular weathering.

Regardless of how the materials are formed, for engineering purposes, they are generally broken into the categories of bedrock, gravel, sand, silt, clay or a combination of these. The behavioral characteristics of any soil are generally based on the properties of the major constituent(s). Listed below are some properties associated with each of these material types.

1. Sand - The behavior of sand depends on grain size, gradation, density and water conditions. Sand scours easily, so foundations on sand must be protected in areas subject to scour.

2. Silt - This is a relatively poor foundation material. It scours and erodes easily and causes large volume changes when subject to frost.

3. Clay - This material needs to be investigated very carefully for use as a bearing material. Long-term consolidation may be an issue.

4. Bedrock - This is generally the best foundation material. Wisconsin has shallow weathered rock in many areas of the state. Weathered granite and limestone become sands. Shale and sandstone tend to weather more on exposure.

5. Mixture of soils - This is the most common case. The soil type with predominant behavior has the controlling name. For example, a soil composed of sand and clay is called sandy clay if the clayey fraction controls behavior.

10.4 Site Investigation Report

The following is a sample of a Site Investigation Report. The subsurface exploration drawing is also submitted with the report.

CORRESPONDENCE/MEMORANDUM ___________________State of Wisconsin

DATE: February 5, 2008

TO: Bill Niemi, P.E.
Southwest Region, Madison Office Soils and Materials Engineer

FROM: Jeffrey D Horsfall, P.E.
Geotechnical Engineer

SUBJECT: **Site Investigation Report**
Project I.D. 1390-04-01
B-28-146/147
STH 26 over Crawfish River
Jefferson County

Attached is the Site Investigation Report for the above project.

Please call if you have any questions.

Attachments

cc: Southwest Region, Madison Office (via e-mail)
Bureau of Structures, Structures Design (via e-mail)
Tom Zalewski, Earth Tech (via e-mail)
Central Office Files
Geotechnical File (original)

Site Investigation Report
Project I.D. 1390-04-01
Structure B-28-146/147
STH 26 over the Crawfish River
Jefferson County
February 5, 2008

1. GENERAL

The project is located on STH 26 over the Crawfish River, west of the city of Jefferson, Jefferson County. The two new structures will each have three spans and are part of the Jefferson Bypass project. The proposed base of the south abutments will require approximately 7 feet of fill, the piers will require 2 feet to 12 feet of excavation and the north abutments will require approximately 8 feet to 11 feet of excavation. Topography near the proposed structure is the lowlands and the Crawfish River. The project will be constructed using LRFD design for pile-supported substructures. The plans are in English units.

The Southwest Region, Madison Office requested that the Geotechnical Unit evaluate the foundation support for the proposed bridges. The following report presents the results of the subsurface investigation, the design evaluation, the findings, the conclusions and the recommendations.

2. SUBSURFACE CONDITIONS

The Geotechnical Section drill crews completed eight borings near the proposed structure. Samples were collected in the borings with a method conforming to AASHTO T-206, Standard Penetration Test, in May and June 2007, using an automatic hammer. The purpose of the borings was to define subsurface soil conditions at this location. Soil textures in the boring logs were field identified by the drillers. Attachment 1 presents tables showing the summaries of subsurface conditions logged in the borings at this site and at the time of drilling for the structures. Attachment 2 presents the results of the soils laboratory tests. Attachment 3 presents figures that illustrate the boring locations and graphical representations of the boring logs. The original borings logs are available at the Central Office Geotechnical Unit and will be made available upon request.

The following describes the subsurface conditions in the eight borings:

South Side Land Borings (B-1, B-3 and B-6)
1.5 feet to 5.0 feet of topsoil or black silt, overlying
4.5 feet to 9.5 feet of gray, soft, silt, trace clay, trace organics, overlying
2.5 feet to 8.5 feet of gray to brown, loose to firm, fine to medium, sand, overlying
52.0 feet to 65.0 feet of gray, medium hard, clay, some silt, overlying
Gray, firm to very dense, fine to course sand, some silt, some gravel and boulders (B-3 had a layer red silt at the base of the boring)

Water Borings (B-2, B-4 and B-5)
2.0 feet to 6.0 feet of water, overlying
0.0 feet to 6.0 feet of gray, very soft, silt, some sand, little organics, overlying
62.0 feet to 65.0 feet of gray, medium hard, clay, trace silt, overlying
Gray, firm to very dense, fine to course, sand, some silt, some gravel

North Side Land Borings (B-8 and B-7)
1.0 feet to 3.0 feet of topsoil or light brown sand or silt, overlying
12.0 feet to 14.0 feet of light brown to gray, loose, sand and silt, mottled, overlying
2.0 feet to 3.0 feet of light brown, firm, fine to medium, sand, overlying
12.0 feet to 15.5 feet of gray, soft, silt, little sand, overlying
55.0 feet to 59.5 feet of gray, medium hard, clay, trace silt, overlying
Gray, loose to very dense, sand, little silt, some gravel, trace boulders

The approximate water level at the time of drilling varied from elevation 773.5 feet to elevation 779.2 feet for the south side land borings, from elevation 777.0 feet to 778.0 feet in the Crawfish River and from 794.0 feet to 796.6 feet for the north side land borings. In addition, two of the borings (B-2 and B-4) experienced artesian conditions with a maximum water pressure of 7 feet above the level of the Crawfish River.

3. ANALYSIS ASSUMPTIONS

The foundation analyses are separated into a shallow foundation (spread footings) or a deep foundation (piling support). The analyses used the following assumptions:

Shallow Foundation (Allowable Stress Design)
1. The water level ranged from elevation 773.5 feet to elevation 796.6 feet.
2. The base of the foundations are at following elevations:

Table 1: Foundation Elevations

Substructure	B-28-146	B-52-147
South Abutment	790.7	792.5
Pier 1	771.0	771.2
Pier 2	769.2	768.6
North Abutment	786.9	788.9

3. The width of the footing is 6 feet.
4. The factor of safety is 3.0 for the allowable bearing capacity.

Deep Foundation (Load and Reduction Factor Design)
1. Soil pressures for displacement piles are based upon a 10 3/4-inch diameter cast-in-place pile.
2. The water level ranged from elevation 773.5 feet to elevation 796.6 feet.

3. Ultimate bearing capacity determined using the FHWA computer program DRIVEN.
4. The drivability evaluation used the computer program GRLWEAP.
5. The lateral capacities of the CIP piles are determined using the computer program LPILE.

Table 3 presents the design shear strength and unit weights used in the analysis. The values are based upon, empirical formulas for angle of internal friction using the Standard Penetration Test results and the effective overburden pressure and for cohesion using the pocket penetrometer tests results.

4. RESULTS OF ANALYSIS

Shallow Foundation
The estimated allowable bearing capacity at the base of the substructures (see Table 1) ranged from 1,700 psf for the fill at the south abutment with a 2H:1V end slope to 1,900 psf to 5,300 psf for the silt and clay below the piers and north abutment. Table 2 presents the results of the allowable bearing capacity for the soils at the base of the substructure footings.

Table 2: Allowable Bearing Capacity (psf)

Substructure	B-28-146	B-28-147
South Abutment	1,700 psf	1,700 psf
Pier 1	5,500 psf	1,900 psf
Pier 2	4,900 psf	5,300 psf
North Abutment	2,000 psf	3,000 psf

The estimated settlement from the bridge loads on the shallow foundations will be excessive at the substructures. The estimated settlement at the south abutment from the embankment load will range from 2 inches to 3 inches. The time for settlement would occur over several months in the fine-grained soils.

Deep Foundation
Table 3 shows the estimated skin friction and end bearing values.

Drivability
The drivability evaluation used a Delmag D-16-32 diesel hammer to determine if the pile would be overstressed during pile installation. The results of the evaluation indicated that a 10 3/4-inch CIP pile should have the minimum pile thickness increased from 0.219-inches to 0.365-inches for the pile not to be overstressed during installation.

Lateral Pile Load Capacity
The lateral capacity of the 10 3/4-inch CIP piles is 8 kips for the front row and 12 kips at the back row at the south abutment and 14 kips for both rows of piles at the north abutment. The proposed abutment type is an A4.

Table 3: Soil Parameters and Foundation Capacities					
Soil Description	**Friction Angle (degrees)**	**Cohesion (psf)**	**Unit Weight (pcf)**	**Nominal Skin Friction[1] (psf)**	**Nominal End Bearing[1] (psf)**
B-28-146 South Abutment (B-1)					
Fill, granular (replaces excavated 2' of topsoil) (Elevation 790.7 ft – 781.4 ft)	32	0	120	160	14,100
Silt, gray, trace clay, trace organics (Elevation 781.4 ft – 771.9 ft)	0	500	110	440	4,500
Sand, gray, fine to medium (Elevation 771.9 ft – 769.4 ft)	31	0	118	470	20,700
Silt, gray, trace silt (Elevation 769.4 ft – 755.4 ft)	0	2,000	120	1,510	18,000
Clay, gray, some silt (Elevation 755.4 ft – 723.4 ft)	0	2,250	123	1,570	20,200
Clay, gray, some silt (Elevation 723.4 ft – 704.4 ft)	0	2,500	125	1,520	22,500
Limestone Boulder (Elevation 704.4 ft – 703.4 ft)	NA	NA	NA	NA	NA
Sand, fine to medium, some silt, some gravel (Elevation 703.4 ft – 695.4 ft)	35	0	135	2,270	107,700
Boulder (Elevation 695.4 ft – 694.4 ft)	NA	NA	NA	NA	NA
Sand, gray, fine to medium, some silt, some gravel (Elevation 694.4 ft and below)	0	36,000	140	NA	Refusal

1. Skin friction and end bearings values are nominal values and have not been modified by a resistance factor.
2. NA - not applicable
3. Refusal, steel pile will obtain capacity in the very dense sand and silt layer.

Table 3: Soil Parameters and Foundation Capacities (Continued)

Soil Description	Friction Angle (degrees)	Cohesion (psf)	Unit Weight (pcf)	Nominal Skin Friction[1] (psf)	Nominal End Bearing[1] (psf)
B-28-147 South Abutment (B-3)					
Fill, granular (replaces excavated 1.5' of black silt) (Elevation 792.5 ft – 783.7 ft)	32	0	120	150	13,300
Silt, light brown (Elevation 783.7 ft – 779.2 ft)	0	1,000	115	820	9,000
Sand, gray to brown, fine to course (Elevation 779.2 ft – 770.7 ft)	31	0	118	460	20,700
Clay, gray to brown, trace silt (Elevation 770.7 ft – 750.2 ft)	0	1,000	115	950	9,000
Clay, gray to brown, trace silt (Elevation 750.2 ft – 730.2 ft)	0	1,500	118	1,290	13,500
Clay, gray to brown, trace silt (Elevation 730.2 ft – 705.7 ft)	0	1,750	118	1,430	15,800
Sand, gray, fine to course, some silt, little gravel (Elevation 705.7 ft – 699.7 ft)	30	0	115	1,320	13,300
Silt, gray, trace clay (Elevation 699.7 ft – 695.7 ft)	0	4,500	135	1,250	40,500
Sand, brown, fine to course, some silt, little gravel (Elevation 695.7 ft – 691.2 ft)	35	0	135	2,320	107,700
Sand, brown, fine to course, some silt, little gravel (Elevation 691.2 ft – 680.2 ft)	0	36,000	140	NA	Refusal
Silt, red, some gravel, little clay (Elevation 680.2 ft and below)	0	36,000	140	NA	Refusal

1. Skin friction and end bearings values are nominal values and have not been modified by a resistance factor.
2. NA - not applicable
3. Refusal, steel pile will obtain capacity in the very dense sand and silt layer.

Table 3: Soil Parameters and Foundation Capacities (Continued)

Soil Description	Friction Angle (degrees)	Cohesion (psf)	Unit Weight (pcf)	Nominal Skin Friction[1] (psf)	Nominal End Bearing[1] (psf)
B-28-146 Pier 1 (B-2)					
Silt, gray, some sand, some organics (Elevation 771.0 ft – 770.0 ft)	28	0	105	10	300
Clay, gray, trace silt (Elevation 770.0 ft – 749.0 ft)	0	2,500	125	1,250	22,500
Clay, gray, trace silt (Elevation 749.0 ft – 729.0 ft)	0	2,500	125	1,520	22,500
Clay, gray, trace silt (Elevation 729.0 ft – 708.0 ft)	0	2,250	123	1,570	20,300
Sand, gray, fine to medium, some silt, little gravel (Elevation 708.0 ft – 700.5 ft)	38	0	135	2,170	268,700
Sand and Gravel, fine to course, some silt, boulders (Elevation 700.5 ft and below)	0	36,000	140	NA	Refusal
B-28-147 Pier 1 (B-6)					
Sand, fine to medium, trace shells (Elevation 771.2 ft – 768.5 ft)	30	0	110	10	1,100
Clay, gray (Elevation 768.5 ft – 743.0 ft)	0	2,000	120	1,410	18,000
Clay, gray (Elevation 743.0 ft – 728.0 ft)	0	2,000	120	1,510	18,000
Clay, gray (Elevation 728.0 ft – 708.0 ft)	0	2,250	123	1,570	20,300
Gravel (Elevation 708.0 ft – 704.0 ft)	38	0	135	1,990	268,700
Sand, brown, some silt, little gravel (Elevation 704.0 ft and below)	0	36,000	140	NA	Refusal

1. Skin friction and end bearings values are nominal values and have not been modified by a resistance factor.
2. NA - not applicable
3. Refusal, steel pile will obtain capacity in the very dense sand and silt layer.

Table 3: Soil Parameters and Foundation Capacities (Continued)

Soil Description	Friction Angle (degrees)	Cohesion (psf)	Unit Weight (pcf)	Nominal Skin Friction[1] (psf)	Nominal End Bearing[1] (psf)
B-28-146 Pier 2 (B-4)					
Clay, gray, some silt (Elevation 769.2 ft – 753.0 ft)	0	2,500	125	1,140	22,500
Clay, gray, some silt (Elevation 753.0 ft – 733.0 ft)	0	2,500	125	1,520	22,500
Clay, gray, some silt (Elevation 733.0 ft – 709.0 ft)	0	2,250	123	1,570	20,300
Sand, gray, fine to medium, some silt, little gravel (Elevation 709.0 ft – 705.0 ft)	32	0	120	1,090	33,000
Gravel, fine to course, little sand (Elevation 705.0 ft – 701.0 ft)	38	0	135	2,130	268,700
Sand and Gravel, fine to course, some silt (Elevation 701.0 ft and below)	0	36,000	140	NA	Refusal
B-28-147 Pier 2 (B-5)					
Clay, gray, some silt (Elevation 768.6 ft – 753.0 ft)	0	2,750	128	1,010	24,800
Clay, gray, some silt (Elevation 753.0 ft – 733.0 ft)	0	2,500	125	1,520	22,500
Clay, gray, some silt (Elevation 733.0 ft – 707.5 ft)	0	2,500	125	1,520	22,500
Sand, gray, fine to course, some silt, some gravel (Elevation 707.5 ft – 703.5 ft)	34	0	130	1,360	73,600
Sand and Gravel, gray, fine to course, some silt (Elevation 703.5 ft – 695.0 ft)	40	0	138	2,790	417,800
Sand and Gravel, gray, fine to course, some silt (Elevation 695.0 ft – 689.0 ft)	32	0	120	1,400	33,000
Gravel, gray, fine to course, few boulders (Elevation 689.0 ft and below)	0	36,000	140	NA	Refusal

1. Skin friction and end bearings values are nominal values and have not been modified by a resistance factor.
2. NA - not applicable
3. Refusal, steel pile will obtain capacity in the very dense sand and silt layer.

Table 3: Soil Parameters and Foundation Capacities (Continued)					
Soil Description	**Friction Angle (degrees)**	**Cohesion (psf)**	**Unit Weight (pcf)**	**Nominal Skin Friction[1] (psf)**	**Nominal End Bearing[1] (psf)**
B-28-146 North Abutment (B-8)					
Silt, brown to gray, little sand, mottled (Elevation 786.9 ft – 782.6 ft)	0	1,000	110	800	9,000
Sand, light brown, fine to medium (Elevation 782.6 ft – 780.6 ft)	34	0	125	90	9,900
Silt, gray, little sand (Elevation 780.6 ft – 768.6 ft)	0	1,500	115	1,140	13,500
Clay, gray, little silt (Elevation 768.6 ft – 742.6 ft)	0	2,500	125	1,520	22,500
Clay, gray, little silt (Elevation 742.6 ft – 722.6 ft)	0	1,500	115	1,290	13,500
Clay, gray, little silt (Elevation 722.6 ft – 709.1 ft)	0	2,000	120	1,510	18,000
Sand, gray, some silt, little gravel, trace boulders (Elevation 709.1 ft and below)	0	36,000	140	NA	Refusal

1. Skin friction and end bearings values are nominal values and have not been modified by a resistance factor.
2. NA - not applicable
3. Refusal, steel pile will obtain capacity in the very dense sand and silt layer.

Table 3: Soil Parameters and Foundation Capacities

Soil Description	Friction Angle (degrees)	Cohesion (psf)	Unit Weight (pcf)	Nominal Skin Friction[1] (psf)	Nominal End Bearing[1] (psf)
B-28-147 North Abutment (B-7)					
Silt, brown, trace sand, mottled (Elevation 788.9 ft – 782.0 ft)	0	1,500	115	1,060	13,500
Sand, brown, fine (Elevation 782.0 ft – 779.0 ft)	34	0	125	150	16,800
Silt, gray, trace sand (Elevation 779.0 ft – 763.5 ft)	0	2,000	120	1,380	18,000
Clay, gray, trace silt (Elevation 763.5 ft – 742.0 ft)	0	2,000	120	1,510	18,000
Clay, gray, trace silt (Elevation 742.0 ft – 722.0 ft)	0	2,000	120	1,510	18,000
Clay, gray, trace silt (Elevation 722.0 ft – 712.0 ft)	0	1,500	115	1,290	13,500
Silt, gray, little clay, trace sand and gravel (Elevation 712.0 ft – 708.5 ft)	0	1,000	110	940	9,000
Sand, gray, little silt and gravel (Elevation 708.5 ft – 700.0 ft)	29	0	110	1,010	13,300
Sand, gray, some gravel, little silt (Elevation 700.0 ft – 691.0 ft)	32	0	120	1,470	33,000
Sand, gray, little silt, some gravel (Elevation 691.0 ft and below)	0	36,000	140	NA	Refusal

1. Skin friction and end bearings values are nominal values and have not been modified by a resistance factor.
2. NA - not applicable
3. Refusal, steel pile will obtain capacity in the very dense sand and silt layer.

Lateral Earth Pressure
The lateral earth pressure for the backfill material will exert 36 psf for sandy soils. The backfill material will be granular, free draining and locally available.

Scour
The bridge section will estimate the scour depth based on the soil types located in the riverbed.

5. FINDING AND CONCLUSIONS

The following findings and conclusions are based upon the subsurface conditions and the analysis:

1. The following describes the subsurface conditions in the eight borings:

   South Side Land Borings (B-1, B-3 and B-6)
   1.5 feet to 5.0 feet of topsoil or black silt, overlying
   4.5 feet to 9.5 feet of gray, soft, silt, trace clay, trace organics, overlying
   2.5 feet to 8.5 feet of gray to brown, loose to firm, fine to medium, sand, overlying
   52.0 feet to 65.0 feet of gray, medium hard, clay, some silt, overlying
   Gray, firm to very dense, fine to course sand, some silt, some gravel and boulders (B-3 had a layer red silt at the base of the boring)

   Water Borings (B-2, B-4 and B-5)
   2.0 feet to 6.0 feet of water, overlying
   0.0 feet to 6.0 feet of gray, very soft, silt, some sand, little organics, overlying
   62.0 feet to 65.0 feet of gray, medium hard, clay, trace silt, overlying
   Gray, firm to very dense, fine to course, sand, some silt, some gravel

   North Side Land Borings (B-8 and B-7)
   1.0 feet to 3.0 feet of topsoil or light brown sand or silt, overlying
   12.0 feet to 14.0 feet of light brown to gray, loose, sand and silt, mottled, overlying
   2.0 feet to 3.0 feet of light brown, firm, fine to medium, sand, overlying
   12.0 feet to 15.5 feet of gray, soft, silt, little sand, overlying
   55.0 feet to 59.5 feet of gray, medium hard, clay, trace silt, overlying
   Gray, loose to very dense, sand, little silt, some gravel, trace boulders

2. The approximate water level at the time of drilling varied from elevation 773.5 feet to elevation 779.2 feet for the south side land borings, from elevation 777.0 feet to 778.0 feet in the Crawfish River and from 794.0 feet to 796.6 feet for the north side land borings. In addition, two of the borings (B-2 and B-4) experienced artesian conditions with a maximum water pressure of 7 feet above the level of the Crawfish River.

3. The estimated allowable bearing capacity at the base of the substructures (see Table 1) ranged from 1,700 psf for the fill at the south abutment with a 2H:1V end slope to 1,900 psf to 5,300 psf for the silt and clay below the piers and north abutment. Table 2 presents

the results of the allowable bearing capacity calculations. The calculation utilized a factor of safety of 3.

4. Support of the piles will occur in the very dense silt, weathered bedrock or bedrock. The pile tip elevation will range from 693 feet to 709 feet. The driven pile lengths will depend upon the type of pile hammer used and the actual subsurface conditions encountered.

5. The drivability evaluation used a Delmag D-16-32 diesel hammer to determine if the pile would be overstressed during pile installation. The results of the evaluation indicated that a 10 3/4-inch CIP pile should have the minimum pile thickness increased from 0.219-inches to 0.365-inches for the pile not to be overstressed during installation.

6. The lateral capacity of the 10 3/4-inch CIP piles is 8 kips for the front row and 12 kips at the back row at the south abutment and 14 kips for both rows of piles at the north abutment. The proposed abutment type is an A4.

6. RECOMMENDATIONS

The following recommendations are based upon the findings and conclusions:

1. The recommended support system for the abutments and piers are 10 3/4-inch diameter cast in place piles driven to a “Required Driving Resistance” of 186 tons using modified Gates dynamic formula. These values use a “Factored Axial Compression Resistance” of 74.5 tons and a Resistance Factor of 0.4 for the modified Gates dynamic formula.

   Table 4 presents the estimated pile tip elevations for the substructures. The actual pile lengths compared to the design pile lengths maybe shorter, because the piles may achieve bearing in the dense sand or silt layers above the very dense sand and gravel layers.

Table 4: Estimated 10 3/4-inch diameter CIP Pile Tip Elevations		
Substructure	B-28-146	B-28-147
South Abutment	703 feet	693 feet
Pier 1	708 feet	708 feet
Pier 2	705 feet	703 feet
North Abutment	709 feet	694 feet

2. The drivability evaluation indicated that the driving stresses require the thickness of the 10 3/4-inch diameter CIP piles should be increased from 0.219-inches to 0.365-inches.

3. Flush mounted end plates should be used at the end of the 10 3/4-inch diameter cast in place pile to reduce the potential for loss of skin friction resistance because of the artesian conditions.

4. Pile spacing should be as great as possible to reduce the effect of densification of the soils during installation of cast in place piles within the cofferdam.

5. The settlement of the subsurface soils caused by the construction of the south abutment approach embankments will cause excessive down drag loads on the piles. Therefore, the plans should provide details for excavating the topsoil and silt and backfilling with granular fill prior to installation of the embankments and piling at the south abutments. The limits of the excavation should be based on a 1.5H:1V slope from the east and west subgrade shoulder points and from the front of the south abutments and the back of the approach slabs. The estimated depth of the excavation should range from 11 feet (east end) to 7 feet (west end).

   An alternative option would be to surcharge the south abutment areas prior to construction of the bridges. The estimated time for the consolidation of the compressible materials would be approximately 6 months.

6. Granular 1 backfill should be used behind the abutments.

Attachment 1
Tables of Subsurface Conditions

B-28-146/147 South Abutment							
B-1 Station 559+10 45.2 feet right of STH 26 NB RL				**B-3 Station 559+19 105.3 feet left of STH 26 NB RL**			
Top of Soil Layer Elevation (feet)	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count**	**Top of Soil Layer Elevation (feet)**	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count**
783.4	Topsoil			785.2	Silt, black		
781.4	Silt, gray, trace clay, trace organics Qp=0.25	5	9	783.7	Silt, light brown Qp=1.0	7	13
771.9	Sand, gray, fine to medium	11	18	779.2	Sand, gray to brown, fine to course	9	16
769.4	Silt, gray, trace clay Qp=2.0 – 2.25	8,11, 11	12,16, 15	770.7	Clay, brown to gray, trace silt Qp=0.75 – 3.0	8,6,6, 11	13,9, 8,15
755.4	Clay, gray, some silt Qp=1.5 – 2.5	11,9, 13,11, 13,10	15,11, 16,13, 15,11	750.2	Clay, brown to gray, trace silt Qp=0.25 – 2.5	10,11, 8,12, 11	13,14, 9,14, 12
723.4	Clay, gray, some silt Qp=1.75 – 3.0	12,20, 19,20	13,20, 19,19	730.2	Clay, brown to gray, trace silt Qp=1.0 – 2.5	7,14, 14,15	7,14, 14,15
704.4	Limestone Boulder			705.7	Sand, gray, fine to course, some silt, little gravel	12	11
703.4	Sand, fine to medium, some silt, some gravel	51,35	47,31	699.7	Silt, gray, trace clay Qp=4.5	18	16
695.4	Boulder			695.7	Sand, brown, fine to course, some silt, little gravel	45	40
694.4	Sand, gray, fine to medium, some silt, some gravel	100/5” 100/5” 100/4”	81,84, 81	691.2	Sand, brown, fine to course, some silt, little gravel	94, 100/5” 72	81,83, 58
682.4	EOB			680.2	Silt, red, some gravel, little clay		
				679.2	EOB		

1. Blow counts are corrected for SPT hammer efficiency and overburden pressure.
2. First elevation is the surface elevation for the boring
3. Qp = Unconfined compression strength as determined by a pocket penetrometer, tons/ft^2
4. EOB is the end of boring

B-28-146/147 Pier 1							
B-2 Station 560+77 18 feet right of STH 26 NB RL				**B-6 Station 560+54 79.1 feet left of STH 26 NB RL**			
Top of Soil Layer Elevation (feet)	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count**	**Top of Soil Layer Elevation (feet)**	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count**
779.0	Barge			783.0	Silt, black, trace clay		
778.0	Water			778.0	Silt, brown, trace sand, trace fibers Qp=0.5 – 1.0	4	8
776.0	Silt, gray, some sand, some organics	0,6,4	0,16,9	773.5	Sand, fine to medium, trace shells	5	8
770.0	Clay, gray, trace silt Qp=2.0 – 2.75	8,14, 8,12,8	17,28, 14,20, 12	768.5	Clay, gray Qp=0.5 – 2.75	8,9,7, 9,10,8	12,13, 9,12, 12,9
749.0	Clay, gray, trace silt Qp=1.75 – 2.5	10,13, 10,14	14,18, 13,18	743.0	Clay, gray Qp=1.25 – 2.25	12,10, 10	14,11, 11
729.0	Clay, gray, trace silt Qp=1.75 – 3.0	12,13, 18,16	14,15, 20,17	728.0	Clay, gray Qp=1.5 – 2.75	13,11, 13	13,11, 13
708.0	Sand, gray, fine to medium, some silt, little gravel	95,67	99,67	708.0	Gravel	100/5”	94
700.5	Sand and Gravel, fine to course, some silt, some boulders ARTESIAN	100/3” 100/2”	97,94	704.0	Sand, brown, some silt, little gravel	72, 100/2” 100/1” 100/1”	65,88, 85,82
694.0	EOB			688.9	EOB		

1. Blow counts are corrected for SPT hammer efficiency and overburden pressure.
2. First elevation is the surface elevation for the boring
3. Qp = Unconfined compression strength as determined by a pocket penetrometer, tons/ft^2
4. EOB is the end of boring

B-28-146/147 Pier 2							
B-4 Station 562+23 **19 feet right of STH 26 NB RL**				**B-5 Station 562+20** **79 feet left of STH 26 NB RL**			
Top of Soil Layer Elevation (feet)	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count**	**Top of Soil Layer Elevation (feet)**	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count**
778.0	Barge			778.0	Barge		
777.0	Water			777.0	Water		
771.0	Clay, gray, some silt Qp=1.75 – 3.0	11,13, 12,13, 12,16	32,32, 26,27, 23,28	773.0	Clay, gray, some silt Qp=2.75 – 3.25	8,15, 11,14	47,34, 21,24
753.0	Clay, gray, some silt Qp=2.25 – 2.5	16,14, 17,15	26,21, 24,20	753.0	Clay, gray, some silt Qp=2.0 – 2.75	14,13, 13,13	22,20, 18,17
733.0	Clay, gray, some silt Qp=1.75 – 2.25	17,14, 16,14, 26	22,17, 19,18, 29	733.0	Clay, gray, some silt Qp=2.0 – 2. 5	16,14, 17,18, 18	21,17, 20,20, 20
709.0	Sand, gray, fine to medium, some silt, little gravel	22	23	707.5	Sand, gray, fine to medium, some silt, some gravel	27	31
705.0	Gravel, fine to course, little sand	35	36	703.5	Sand and Gravel, gray, fine to course, some silt	106, 43	109, 42
701.0	Sand and Gravel, fine to course, some silt ARTESIAN	120, 134, 107	119, 129, 98	695.0	Sand, gray, fine to medium, some silt, some gravel	19	18
686.5	EOB			689.0	Gravel, gray, fine to course, few boulders	100/5” 100/5” 100/3”	93,89, 87
				678.0	EOB		

1. Blow counts are corrected for SPT hammer efficiency and overburden pressure.
2. First elevation is the surface elevation for the boring
3. Qp = Unconfined compression strength as determined by a pocket penetrometer, tons/ft^2
4. EOB is the end of boring

Site Investigation Report
Structure B-28-146/147
Attachment 1

B-28-146/147 North Abutment							
B-8 Station 563+19 45.0 feet right of STH 26 NB RL				**B-7 Station 563+19 105.0 feet left of STH 26 NB RL**			
Top of Soil Layer Elevation (feet)	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count**	**Top of Soil Layer Elevation (feet)**	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count**
797.6	Sand, light brown, some silt			797.0	Topsoil		
				796.0	Silt, light brown		
796.6	Silt, brown to gray, little sand, mottled Qp=1.25	7,6	13,9	794.0	Sand, light brown, some silt, mottled	9	16
				789.0	Silt, brown, tr. sand, mottled Qp=1.5	9	15
782.6	Sand, light brown, fine to medium	27	37	782.0	Sand, brown, fine	16	25
780.6	Silt, gray, little sand Qp=1.25 – 3.0	6,8	8,10	779.0	Silt, gray, trace sand Qp= 0.5 – 2.5	5,8,11	7,11, 15
768.6	Clay, gray, little silt Qp=1.5 – 2.75	11,10 10,9	13,12, 11,9	763.5	Clay, gray, trace silt Qp=1.75 – 2.25	13,12, 14,11	16,15, 16,12
742.6	Clay, gray, little silt Qp=1.25 – 2.2	9,10, 12,10	9,10, 11,9	742.0	Clay, gray, trace silt Qp=1.25 – 2.5	11,13, 11,11	12,14, 11,11
722.6	Clay, gray, little silt Qp=2.0 – 2.25	10,11, 16	9,10, 14	722.0	Clay, gray, trace silt Qp=1.75 – 2.0	11,11	11,10
709.1	Sand, gray, some silt, little gravel, trace boulders	92, 100/3” 100/4”	77,82, 79	712.0	Silt, gray, little clay, tr. sand and gravel Qp=0.75 – 1.2	10	9
697.1	EOB			708.5	Sand, gray, little silt and gravel	6,12	5,10
				700.0	Sand, gray, some gravel, little silt	32,21	27,17
				691.0	Sand, gray, some gravel, little silt	100/1” 100/0”	80,77
				683.0	EOB		

1. Blow counts are corrected for SPT hammer efficiency and overburden pressure.
2. First elevation is the surface elevation for the boring
3. Qp = Unconfined compression strength as determined by a pocket penetrometer, tons/ft^2
4. EOB is the end of boring

Attachment 2
Soils Laboratory Test Results

REPORT ON TESTS ON SOILS
EL140(L) 1-78

Wisconsin Department of Transportation
Div. of Transportation Infrastructure Development
Bureau of Highway Construction
3502 Kinsman Blvd., Madison, Wi 53704 - 2507

TEST NUMBER
230-6-07

	MO.-DAY-YR.			PROJECT ID	TEST CODE		QUANTITY
	6	12	07	1390-04-01	**190**	**2**	1 tube

County: Jefferson	Project Name: STH 26 over Crawfish River
Contractor:	
Material: Soil	
Source:	
Tests Requested By: Foundation & Pavements	
Submitted by: Jeff Horsfall	Date: 6/4/07

BORING NUMBER	8	8	8			
SAMPLE NUMBER	bot	mid	top			
DEPTH, FT.	39.5-42.0	39.5-42.0	39.5-42.0			
MECHANICAL ANALYSIS (AASHTO T-88)						
Boulders (Ret. 3")						
Gravel (Pass 3" - Ret. #10)						
Coarse Sand (Pass #10 - Ret. #40)						
Fine Sand (Pass #40 - Ret. #200)		0				
Silt (Pass #200 - Ret. 0.002mm)		54				
Clay (Pass 0.002mm)		46				
LIQUID LIMIT (AASHTO T-89)		37				
PLASTICITY INDEX (AASHTO T-90)		20				
UNIFIED CLASSIFICATION (ASTM D 2487)		CL				
AASHTO CLASSIFICATION (AASHTO M-145)		A-6(21)				
LOSS ON IGNITION, % (AASHTO T-267)						
MOISTURE, % (AASHTO T-265)		21.4				
UNCONFINED COMPRESSION TEST (AASHTO T-208)						
MOISTURE CONTENT, %	23.0		23.2			
UNIT WEIGHT, PCF	104.7		104.9			
PENETRATION RESISTANCE, TSF	1.5	3.25	1.5			
UNCONFINED COMPRESSION, PSF	5506		5150			
CONSOLIDATION TEST (AASHTO T-216)						
MOISTURE BEFORE, %						
MOISTURE AFTER, %						
COMPRESSION INDEX						
DIRECT SHEAR TEST (AASHTO T-236)						
MOISTURE CONTENT, %						
ANGLE OF INTERNAL FRICTION, DEGREES						
COHESION, PSF						

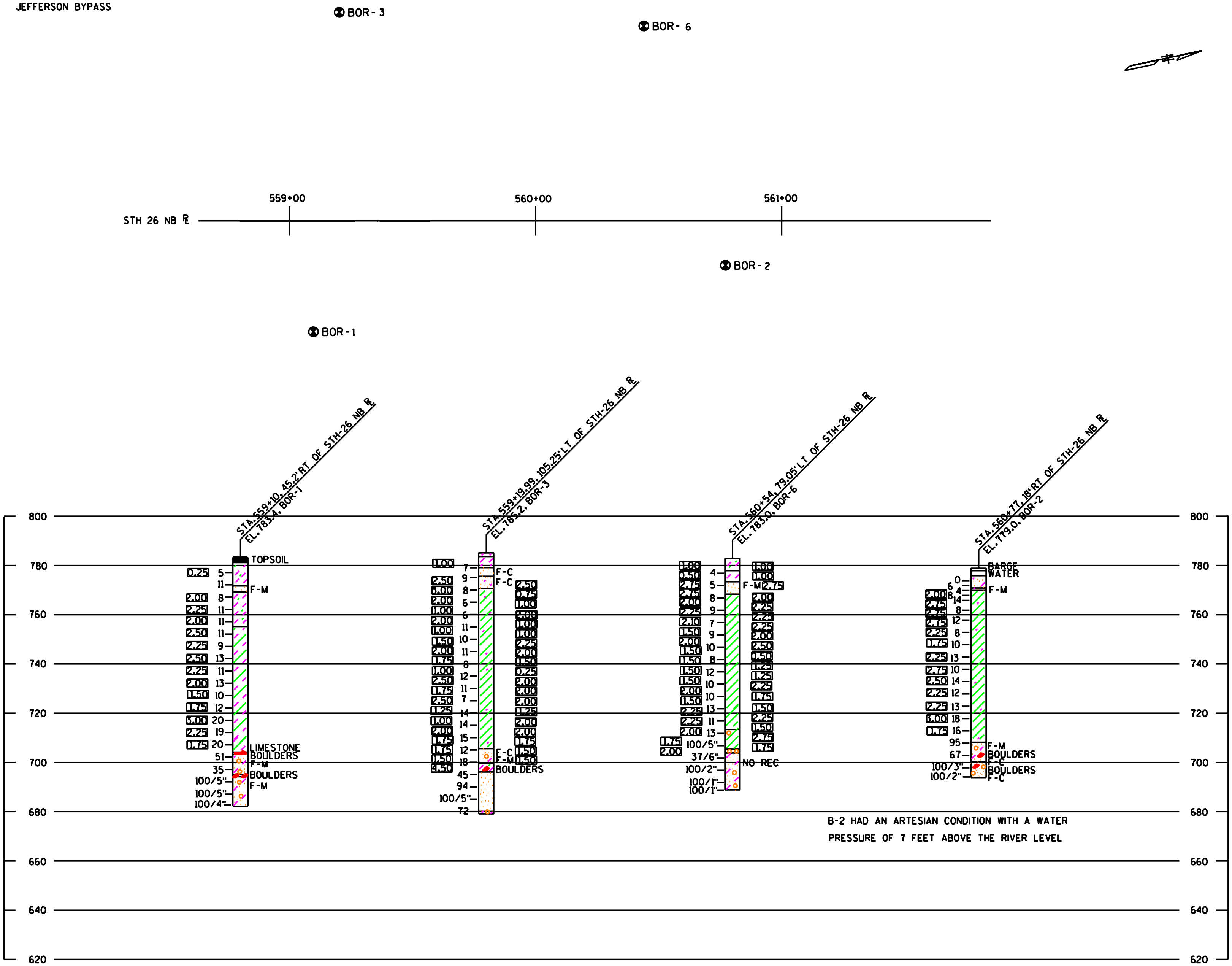
Remarks

DISTRIBUTION: Geotechnical Unit
District

By WRK

Attachment 3
Bridge Figures

STH 26 OVER CRAWFISH RIVER
JEFFERSON BYPASS



STATE PROJECT NUMBER

ABBREVIATIONS

F — FINE M — MEDIUM C — COARSE
WS — WEATHERED SO — SOUND

MATERIAL SYMBOLS

TOPSOIL SILT SANDSTONE
SAND PEAT LIMESTONE
GRAVEL CLAY IGNEOUS ROCK

LEGEND OF PROBING

PROBING NO.
STA.
ELEVATION
7 AVERAGE BLOWS PER FOOT
REFUSAL 95/6

95/6=95 BLOWS FOR 6" PENETRATION PROBING TAKEN WITH A 350# WT. FALLING 18" ON A 2" O.D. POINT.

LEGEND OF BORING

BORING NO.
STA.
ELEV.
UNCONFINED STRENGTH 7.7
BLOWS PER FT. USING 140# WT. FALLING 30"
WASH SAMPLE
SHELBY TUBE — S.T.
GROUND WATER ELEVATION
NO GROUND WATER OBSERVED ABOVE THIS ELEVATION
SANDY GRAVEL
F. BOULDERS OR COBBLES
SAND
SILTY CLAY
SO LIMESTONE

UNLESS OTHERWISE SPECIFIED, THE BLOWS PER FOOT AT THE LOCATIONS INDICATED ARE BASED ON DRIVING A 2" O.D. X 1.4" I.D. SPLIT SPOON SAMPLER WITH A 140# HAMMER HAVING A FREE FALL OF 30". THE BLOW COUNT IS TAKEN IN UNDISTURBED SOIL IMMEDIATELY BELOW A CASED OR OPEN HOLE ELIMINATING SIDE FRICTION ON THE DRIVE PIPE.

SUBSURFACE EXPLORATION FOR FOUNDATION DESIGN AND BIDDERS INFORMATION

TO OBTAIN RELATIVE DATA CONCERNING THE CHARACTER OF MATERIAL IN AND UPON WHICH THE FOUNDATION MIGHT BE BUILT, BORINGS AND/OR SOUNDINGS WERE MADE AT POINTS APPROXIMATELY AS INDICATED ON THIS DRAWING. THE DATA PRESENTED HEREIN REPRESENTS THE FINDINGS OF THE SUBSURFACE EXPLORATIONS MADE. HOWEVER, BECAUSE THE DEPTHS INVESTIGATED ARE LIMITED AND THE AREA OF THE BORINGS AND/OR SOUNDINGS IS VERY SMALL IN RELATION TO THE ENTIRE AREA, THE WISCONSIN DEPARTMENT OF TRANSPORTATION DOES NOT WARRANT CONDITIONS BELOW THE DEPTHS INVESTIGATED OR THAT THE CLASSIFICATION OF MATERIAL ENCOUNTERED IN THESE INVESTIGATIONS IS NECESSARILY TYPICAL OF THE ENTIRE SITE.

NO.	DATE	REVISION	BY

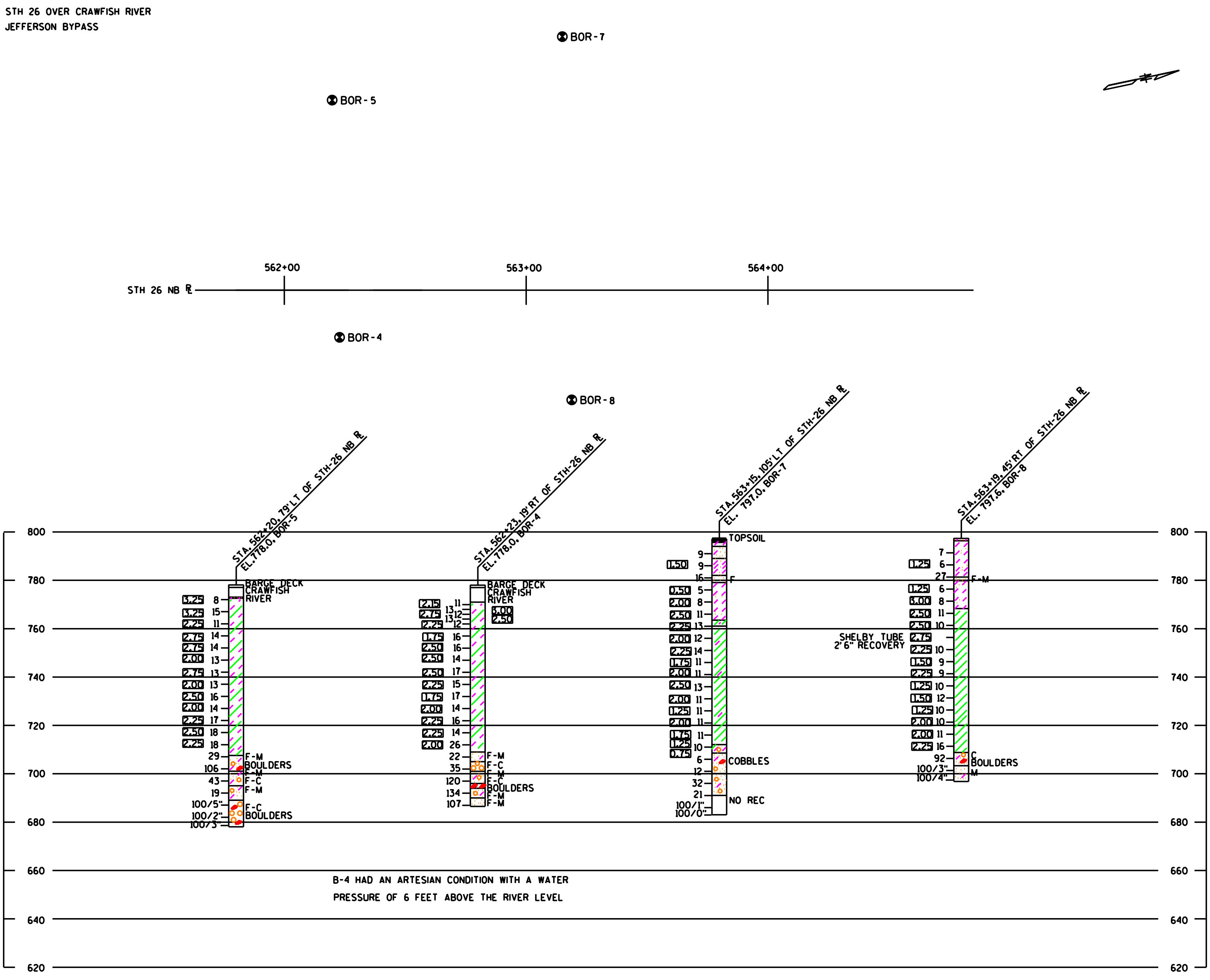
STATE OF WISCONSIN
DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN SECTION

STRUCTURE B-28-146/147A

DRAWN BY L.H PLANS CK'D.

SUBSURFACE EXPLORATION

SHEET 3



STATE PROJECT NUMBER

ABBREVIATIONS

F — FINE M — MEDIUM C — COARSE
WS — WEATHERED SO — SOUND

MATERIAL SYMBOLS

TOPSOIL SILT SANDSTONE
SAND PEAT LIMESTONE
GRAVEL CLAY IGNEOUS ROCK

LEGEND OF PROBING

PROBING NO.
STA.
ELEVATION
95/6=95 BLOWS FOR 6" PENETRATION
PROBING TAKEN WITH A 350# WT. FALLING 18" ON A 2" O.D. POINT.
7 AVERAGE BLOWS PER FOOT
REFUSAL 95/6

LEGEND OF BORING

BORING NO.
ELEV. STA.
UNCONFINED STRENGTH 7.7 7
SANDY GRAVEL
BLOWS PER FT. USING 140# WT. FALLING 30"
F. BOULDERS OR COBBLES
WASH SAMPLE
SAND
SHELBY TUBE — S.T.
GROUND WATER ELEVATION
SILTY CLAY
NO GROUND WATER OBSERVED ABOVE THIS ELEVATION
SO LIMESTONE

UNLESS OTHERWISE SPECIFIED, THE BLOWS PER FOOT AT THE LOCATIONS INDICATED ARE BASED ON DRIVING A 2" O.D. X 1.4" I.D. SPLIT SPOON SAMPLER WITH A 140# HAMMER HAVING A FREE FALL OF 30". THE BLOW COUNT IS TAKEN IN UNDISTURBED SOIL IMMEDIATELY BELOW A CASED OR OPEN HOLE ELIMINATING SIDE FRICTION ON THE DRIVE PIPE.

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NO.	DATE	REVISION	BY

STATE OF WISCONSIN
DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN SECTION

STRUCTURE B-28-146/147B

DRAWN BY L.H PLANS CK'D.

SUBSURFACE EXPLORATION

SHEET 3

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11.1 General

11.1.1 Overall Design Process

The overall foundation support design process requires an iterative collaboration to provide cost-effective constructible substructures. Input is required from multiple disciplines including, but not limited to, structural, 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.

- Performance requirements, including deformation (settlement), lateral deflection, global stability and resistance to bearing, uplift, lateral, sliding and overturning forces.

- Ease, time and cost of construction.

- Environmental impact of design and construction.

- Site constraints, including restricted right-of-way, overhead and lateral clearance, construction access, utilities and vibration-sensitive structures.

Based on the items listed above, an assessment is made to determine if shallow or deep foundations are suitable to satisfy site-specific needs. A shallow foundation, as defined in this manual, is one in which the depth to the bottom of the footing is generally less than or equal to twice the smallest dimension of the footing. Shallow foundations generally consist of spread footings but may also include rafts that support multiple columns and typically are the least costly foundation alternative.

Shallow foundations are typically initially considered to determine if this type of foundation is technically and economically viable. Often foundation settlement and lateral loading constraints govern, rather than bearing capacity. Other significant considerations for selection of shallow foundations include requirements for cofferdams, bottom seals, dewatering, temporary excavation support/shoring, over-excavation of unsuitable material, slope stability, available time to dissipate consolidation settlement prior to final construction, scour susceptibility, environmental impacts and water quality impacts. Shallow foundations may not be economically viable when footing excavations exceed 10 to 15 feet below the final ground surface elevation.

When shallow foundations are not satisfactory, deep foundations are considered. Deep foundations can transfer foundation loads through shallow deposits to underlying deposits of more competent deeper bearing material. Deep foundations are generally considered to mitigate concerns about scour, lateral spreading, excessive settlement and satisfy other site constraints.

Common types of deep foundations for bridges include driven piling, drilled shafts, micropiles and augercast piles. Driven piling is the most frequently-used type of deep foundation in Wisconsin. Drilled shafts may be advantageous where a very dense stratum must be penetrated to obtain required bearing, uplift or lateral resistance are concerns, or where obstructions may result in premature driving refusal or where piers need to be founded in areas of shallow bedrock or deep water. A drilled shaft may be more cost effective than driven piling when a drilled shaft is extended into a column and can be used to eliminate the need for a pile footing, pile casing or cofferdams.

Micropiles may be the best foundation alternatives where headroom is restricted or foundation retrofits are required at existing substructures. Micropiles tend to have a higher cost than traditional foundations.

Augercast piles are a potentially cost-effective foundation alternative, especially where lateral loads are minimal. However, restrictions on construction quality control including pile integrity and capacity need to be considered when augercast piles are being investigated. Augercast piles tend to have a higher cost than traditional foundations.

11.1.3 Cofferdams

At stream crossings, tremie-sealed cofferdams are frequently used when footing concrete is required to be placed below the surrounding water level.. The tremie-seal typically consists of a plain-cement concrete slab that is placed underwater (in the wet), within a closed-sided cofferdam that is generally constructed of sheetpiling. The seal concrete is placed after the excavation within the cofferdam has been completed to the proper elevation. The seal has three main functions: allowing the removal of water in the cofferdam so the footing concrete can be placed in the dry; serving as a counterweight to offset buoyancy due to differing water elevations within and outside of the cofferdam; and minimizing the possible deterioration of the excavation bottom due to piping and bottom heave. Concrete for tremie-seals is permitted to be placed with a tremie pipe underwater (in-the-wet). Footing concrete is typically required to be placed in-the-dry. In the event that footing concrete must be placed in-the-wet, a special provision for underwater inspection of the footing subgrade is required.

When bedrock is exposed in the bottom of any excavation and prior to placement of tremie concrete, the bedrock surface must be cleaned and inspected to assure removal of loose debris. This will assure good contact between the bedrock and eliminate the potential consolidation of loose material as the footing is loaded.

Cofferdams need to be designed to determine the required sheetpile embedment needed to provide lateral support, control piping and prevent bottom heave. The construction sequence must be considered to provide adequate temporary support, especially when each row of ring struts is installed. Over-excavation may be required to remove unacceptable materials at the base of the footing. Piles may be required within cofferdams to achieve adequate nominal bearing resistance. WisDOT has experienced a limited number of problems achieving adequate penetration of displacement piles within cofferdams when sheetpiling is excessively deep in granular material. Cofferdams are designed by the Contractor.

Refer to 13.11.5 for further guidance to determine the required thickness of cofferdam seals and to determine when combined seals and footings are acceptable.

11.1.4 Vibration Concerns

Vibration damage is a concern during construction, especially during pile driving operations. The selection process for the type of pile and hammer must consider the presence of surrounding structures that may be damaged due to high vibration levels. Pile driving operations can cause ground displacement, soil densification and other factors that can damage nearby buildings, structures and/or utilities. Whenever pile-driving operations pose the potential for damage to adjacent facilities (usually when they are located within approximately 100 feet), a vibration-monitoring program should be implemented. This program consists of requiring and reviewing a pile-driving plan submittal, conducting pre-driving and post-driving condition surveys and conducting the actual vibration monitoring with an approved seismograph. A special provision for implementing a vibration monitoring program is available and should be used on projects whenever pile-driving operations or other construction activities pose a potential threat to nearby facilities. Contact the geotechnical engineer for further discussion and assistance, if vibrations appear to be a concern.

11.2 Shallow Foundations

11.2.1 General

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

WisDOT policy item:

Design shallow foundations in accordance with the 4th Edition of the AASHTO *LRFD Bridge Design Specifications for Highway Bridges*. No additional guidance is available at this time.

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

11.2.2 Footing Design Considerations

The following design considerations apply to shallow foundations:

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

11.2.2.1 Minimum Footing Depth

Foundation type selection and the preliminary design process require input from the geotechnical and hydraulic disciplines. The geotechnical engineer should provide guidance on the minimum embedment for shallow foundations that takes into consideration frost protection and the possible presence of unsuitable foundation materials. The hydraulic engineer should be consulted to assess scour potential and maximum scour depth for water crossings.

At shallow foundations bearing on rock, it is essential to obtain a proper connection to sound rock. Sometimes it is not possible to obtain deep footing embedment in granite or similar hard rock, due to the difficulty of rock removal.

11.2.2.1.1 Scour Vulnerability

Scour is a hydraulic erosion process caused by flowing water that lowers the grade of a water channel or riverbed. For this reason, scour vulnerability is an essential design consideration for shallow foundations. Scour can undermine shallow foundations or remove sufficient overburden to redistribute foundation forces, causing foundation displacement and detrimental stresses to structural elements. Excessive undermining of a shallow foundation leads to gross deformation and can lead to structure collapse.

Scour assessment will require streambed sampling and gradation analysis to define the median diameter of the bed material, D_{50}. Specific techniques for scour assessment, along with a detailed discussion of scour analysis and scour countermeasure design, are presented in the following publications:

- HEC 18 – *Evaluating Scour at Bridges*, 4th Edition
- HEC 20 – *Stream Stability at Highway Structures*, 3rd Edition
- HEC 23 – *Bridge Scour and Stream Instability Countermeasures - Experience, Selection and Design Guidance*, 2nd Edition

Foundations for new bridges and structures located within a stream or river should be located at an elevation below the maximum scour depth that is identified by the hydraulics engineer. In addition, the foundation should be designed deep enough such that scour protection is not required. If the maximum calculated scour depth elevation is below the practical limits for a shallow foundation, a deep foundation system should be used to support the structure.

11.2.2.1.2 Frost Protection

Shallow foundation footings must be embedded below the maximum depth of frost potential (frost depth) whenever frost heave is anticipated to occur in frost-susceptible soil and adequate moisture is available. This embedment is required to prevent foundation heave due to volumetric expansion of the foundation subgrade from freezing and/or to prevent settling due to loss of shear strength from thawing.

Frost susceptible material includes inorganic soil that contains at least 3 percent, dry weight, which is finer in size than 0.02 millimeters. Gravel that contains between 3 and 20 percent fines is least susceptible to frost heave. Bedrock is not considered frost susceptible if the bedrock formation is massive, dense and intact below the footing.

Foundation design is usually not governed by frost heave for foundations bearing on clean gravel and sand or very dense till. Frost heave is a concern whenever the water table, static or perched, is located within 5 feet of the freezing plane.

In Wisconsin, the maximum depth of frost potential generally ranges from approximately 4 feet in the southeastern part of the state to 6 feet in the northwestern corner of the state.

WisDOT policy item:

The minimum depth of embedment of shallow foundations shall be 4 feet, unless founded on competent bedrock.

Further discussion about frost protection in the design of bridge abutments and piers is presented in 12.5 and 13.6, respectively.

11.2.2.1.3 Unsuitable Ground Conditions

Footings should bear below weak, compressible or loose soil. In addition, some soil exhibits the potential for changes in volume due to the introduction or expulsion of water. These volumetric changes can be large enough to exceed the performance limits of a structure, even to the point of structural damage. Both expansive and collapsible soil is regional in occurrence. Neither soil type is well suited for shallow foundation support without a mitigation plan to address the potential of large soil volume changes in this soil, due to changes in moisture content. Expansive and collapsible soils seldom cause problems in Wisconsin.

It should be noted that the procedures presented herein for computing bearing resistance and settlement are applicable to naturally occurring soil and are not necessarily valid for conditions of modified ground such as uncontrolled fills, dumps, mines and waste areas. Due to the unpredictable behavior of shallow foundations in these types of random materials, deep foundations which penetrate through the random material, overexcavation to remove the random material, or subgrade improvement to improve material behavior is required at each substructure unit.

11.2.2.2 Tolerable Movement of Substructures Founded on Shallow foundations

The bridge designer shall account for any differential settlement (angular distortion) in the design.

WisDOT policy item:

For design of new bridge structures founded on shallow foundations, the maximum permissible movement is 1 inch of horizontal movement at the top of substructure units and 1.5 inches of total estimated settlement of each substructure unit at the Service Limit State.

The sequence of construction can be important when evaluating total settlement and angular distortion. The effects of embankment settlement, as well as settlement due to structure loads, should be considered when the magnitude of total settlement is estimated. It may be possible to manage the settlements after movements have stabilized, by monitoring movements and delaying critical structural connections such as closure pours or casting of decks that are continuous. Generally project timelines may restrict the time available for soil consolidation. Any project delays for geotechnical reasons must be thoroughly transmitted to, and analyzed by, design personnel.

11.2.2.3 Location of Ground Water Table

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

WisDOT policy item:

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

11.2.2.4 Sloping Ground Surface

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

11.2.3 Settlement Analysis

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

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

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

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

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

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

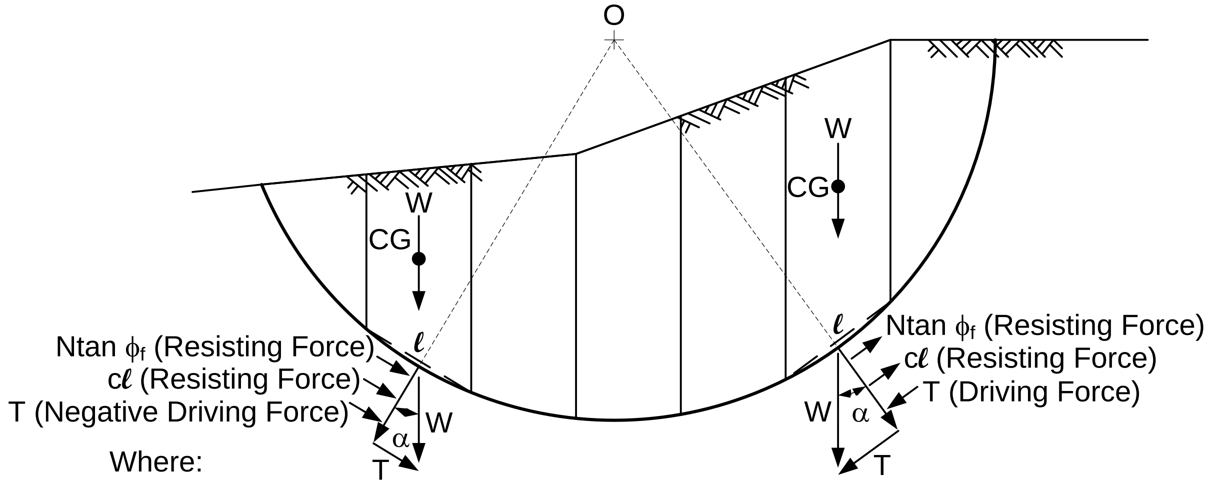
The rate of consolidation is usually of lesser concern for foundations, because superstructure damage will occur once the differential settlements become excessive. Shallow foundations are designed to withstand the settlement that will ultimately occur during the life of the structure, regardless of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

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

11.2.4 Overall Stability

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

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



Where:

CG	=	Center of gravity
c	=	Cohesion along base of slice (psf)
ϕ_f	=	Angle of internal friction of drained soil (degrees)
W	=	Total weight of slice (lb/ft)
N	=	W cos α (lb/ft)
T	=	W sin α (lb/ft)
ℓ	=	length of slice (ft)

Figure 11.2-1
Free Body Diagram for Overall Stability

11.2.5 Footings on Engineered Fills

When shallow foundations are considered for placement on fill, further consideration is required. It is essential to satisfy the design tolerance with regard to total settlement, angular distortion and horizontal movement, including lateral squeeze of the embankment subgrade. The designer must consider the range of probable estimated movement and the impact that this range has on the overall structure performance. The anticipated movement of both new embankment fill and existing embankment materials must be assessed. If shallow foundations are considered, WisDOT requires a thorough subsurface investigation to evaluate settlement of the existing subgrade, including but not limited to continuous soil sampling. WisDOT does not typically place shallow foundations on general embankment fill. WisDOT may consider shallow foundations that are placed on engineered fill, such as that within MSE walls. WisDOT has placed a limited number of shallow foundations on MSE walls for single span bridges. Engineered fill typically consists of high-quality free-draining granular material that is not prone to behavior change due to moisture change, freeze-thaw action, long-term consolidation or shear failure. Engineered fill must also be tightly compacted. On occasion, engineered fill is used in combination with geotextile and/or geogrid to improve shear resistance and overall performance at approach embankments.

If it is not feasible to use a footing to support a sill abutment at the top of slope, it may be feasible to consider a shallow foundation at the bottom of abutment slope to support a full retaining abutment as discussed in 12.2. The increase in stem height will be offset by a reduction in required bridge span length.

11.2.6 Construction Considerations

Shallow foundations require field inspection during construction to confirm that the actual footing subgrade material is equivalent to, or better than, that considered for design. The prepared subgrade should be checked to confirm that the type and condition of the exposed subgrade will provide uniform bearing over the full length or width of footing. The exposed subgrade should be probed to identify possible underlying pockets of soft material that are covered by a thin crust of more competent material. Underlying pockets of soft material and unsuitable material should be over-excavated and replaced with competent material. The structural/geotechnical designer should be contacted if the revised field footing elevations vary by more than one foot lower or three feet higher than the plan elevations, due to differing conditions.

The exposed footing subgrade should be level and stepped, as needed. Stepped shallow foundations may be appropriate when the subsurface conditions vary over the length of substructure unit (footing). For simplicity, planned footing steps should be designated in maximum 4-foot increments. The number and spacing of footing steps is dependent on several factors including, but not limited to, site foundation conditions, temporary excavation support and dewatering requirements, frost and scour depth limitations, constructability, and construction sequence. In general, it is preferred to build uniform step-increments, to simplify construction. Typically the footing with the lowest elevation is constructed first to avoid excavation disturbance of other portions of the footing, as construction continues.

11.2.7 Geosynthetic Reinforced Soil (GRS) Abutment

Geosynthetic Reinforced Soil (GRS) abutments are a type of bridge foundation system typically supporting a single span precast superstructure.. The superstructure is supported on a course-grained soil (gravel) with layers of woven geotextile fabric spaced horizontally from the existing ground, to the base of the slab. The facing is a precast modular block and connected to the woven geotextile fabric. The following reference can be used for design, ‘Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide, Publication Number: FHWA-HRT-11-026’

See 7.1.4.2 for guidance on GRS abutments.

11.3 Deep Foundations

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

The primary functions of a deep foundation are:

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

11.3.1 Driven Piles

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

11.3.1.1 Conditions Involving Short Pile Lengths

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

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

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

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

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

11.3.1.2 Pile Spacing

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

<u>WisDOT policy item:</u>

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

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

11.3.1.3 Battered Piles

Battered piles are used to resist large lateral loads or when there is insufficient lateral soil resistance within the initial 4 to 5 pile diameters of embedment. Battered piles are frequently used in combination with vertical piles. The lateral resistance of battered piling is a function of the vertical load applied to the pile group. Since the sum of the forces at the pile head must equal zero, increasing the number of battered piles does not necessarily increase the lateral load capacity of the pile group. Both the lateral passive resistance of the soil above the footing as well as the sliding resistance developed at the base of footing are generally neglected in design. See the standard details for further guidance when battered piles are used.

Piles are typically battered at 1 horizontal to 4 vertical. Hammer efficiencies must be reduced when piles are battered. Where negative skin friction loads are anticipated, battered piles should not be considered.

11.3.1.4 Corrosion Loss

Piling should be designed with sufficient corrosion resistance to assure a minimum design life of 75 years. Corrosive sites may included those with combinations of organic soils, high water table, man-made coal combustion products or waste materials, and those materials that allow air infiltration such as wood chips. Experience indicates that corrosion is not a practical problem for steel piles driven in natural soil, due primarily to the absence of oxygen in the soil. However, in fill material at or above the water table, moderate corrosion may occur and protection may be required. Concrete piles are prone to deterioration from exposure to excess concentrations of sulfate and/or chloride. Special consideration (including thicker pile shells, heavier pile sections, painting and concrete encasement) should be given to permanent steel piling that is used in areas of northern Wisconsin which are inhabited by corrosion causing bacteria (see FDM Procedure 13-1-15). Typically, WisDOT does not increase pile sections or heavier pile sections to provide corrosion protection outside of these areas.

At potentially corrosive sites, encasement by cast-in-place concrete can provide the required protection for piles extending above the ground surface. All exposed piling should be painted. Additional guidance on corrosion is provided in **LRFD [10.7.5]**.

11.3.1.5 Pile Points

A study was conducted on the value of pile tips (pile points) on steel piles when driving into rock. The results indicated that there was very little penetration difference between the piles driven with pile points and those without. The primary advantages for specifying pile points are for penetrating or displacing boulders, driving through dense granular materials and hardpan layers, and to reduce the potential pile damage in hard driving conditions. Piling can generally be driven faster and in straighter alignment when points are used.

Conical pile points have also been used for round, steel piling (friction and point-bearing) in certain situations. These points can also be flush-welded if deemed necessary.

Standard details for pile points are available from the approved suppliers that are listed in WisDOT's current Product Acceptability List (PAL).

Pile points and preboring are sometimes confused. They are not interchangeable. Pile points can be used to help drive piles through soil that has gravel and/or cobbles or presents other difficult driving conditions. They can also be used to get a good 'bite' when ending piles on sloping bedrock surfaces. Points cannot be used to ensure that piles penetrate into competent bedrock. They may assist in driving through weathered zones of rock or soft rock, but will generally not be effective when penetration into hard rock is desired.

11.3.1.6 Preboring

If embedment into rock is required or minimum pile penetration is doubtful, preboring should be considered. Preboring is required for displacement piles driven into new embankment that are over 10 feet in height. The WisDOT has developed special provisions to provide preboring requirements.

Except for point resistance piles, preboring should be terminated at least 5 feet above the scheduled pile tip elevation. When the pile is planned to be point resistance on rock, preboring may be advanced to plan pile tip elevation. Restrike is not performed when point-bearing piles are founded in rock within prebored holes. Preboring should only be used when appropriate, since many bridge contractors do not own the required construction equipment necessary for this work.

The annular space between the wall of the prebored hole and the pile is required to be backfilled. The annulus in bedrock should be filled with concrete or cement grout after the pile has been installed. Clean sand may be used to backfill the annulus over the depth of soil overburden. Backfill material should be deposited with either a tremie pipe or concrete pump to reduce potential arching (bridging) and assure that the complete depth of hole is filled.

11.3.1.7 Seating

Care must be taken when seating end bearing piles, especially when seating on bedrock with little to no weathered zone. When a pile is firmly seated on rock in prebored holes, pile driving to refusal is not required or recommended, to avoid driving overstress and pile damage. After reaching the predetermined prebore elevation, piles founded in soil are driven with a pile hammer to achieve the specified average penetration or set per blow for the final ten blows of driving.

11.3.1.8 Pile Embedment in Footings

The length of pile embedment in footings is determined based on the type and function of substructure unit and the magnitude of any uplift load.

WisDOT policy item:

Use a minimum 6-inch pile embedment in footings. This embedment depth is considered to result in a free (pinned) head connection for analysis. When the pile embedment depth into the footing is 2.0 feet or greater, the designer can assume a fixed head connection for analysis.

Additional pile embedment is required at some wing walls and at pile-encased substructures, especially where moment connections are required and where cofferdams are not used at stream crossings. Further guidance is provided in 13.6 and in the standard substructure details.

11.3.1.9 Pile-Supported Footing Depth

WisDOT policy item:

Place the bottom of pile-supported footings below the final ground surface at a minimum depth of 2.5 feet for sill abutments, 1.5 feet for sill abutments supported by MSE walls, and 4 feet for piers and other types of abutments.

11.3.1.10 Splices

Full-length piles should be used whenever practical. In no case should timber piles be spliced. Where splices are unavoidable, their number, location and details must be approved by WisDOT prior to pile splicing.

Splice details are shown on Wisconsin bridge plan standards for Pile Details. Splices are designed to develop the full strength of the pile section. Splices should be watertight for CIP concrete piles. Mechanical splice sleeves can be used to join sections of H-pile and pipe pile at greater depth where flexural resistance is not critical. Steel piling 20 feet or less in length is to be furnished in one unwelded piece. Piling from 20 to 50 feet in length can have two shop or field splices, and piling over 50 feet in length can be furnished with up to a maximum of four splices, unless otherwise stated in the project plan documents.

11.3.1.11 Painting

Normally, WisDOT paints all exposed sections of piling. This typically occurs at exposed pier bents.

11.3.1.12 Selection of Pile Types

The selection of a pile type for a given foundation application is made on the basis of soil type, stability under vertical and horizontal loading, long-term settlement, required method of pile installation, substructure type, cost comparison and estimated length of pile. Frequently more than one type of pile meets the physical and technical requirements for a given site. The performance of the entire structure controls the selection of the foundation. Primary considerations in choosing a pile type are the evaluation of the foundation materials and the selection of the substratum that provides the best foundation support.

Piling is generally used at piers where scour is possible, even though the streambed may provide adequate support without piling. In some cases, it is advisable to place footings at greater depths than minimum and specify a minimum pile penetration to guard against excessive scour beneath the footing and piling. Shaft resistance (skin friction) within the maximum depth of scour is assumed to be zero. When a large scour depth is estimated, this area of lost frictional support must be taken into account in the pile driving operations and capacities.

Subsurface conditions at the structure site also affect pile selection and details. The presence of artesian water pressure, soft compressible soil, cobbles and/or boulders, loose/firm uniform sands or deep water all influence the selection of the optimum type of pile for deep foundation support. For instance, WisDOT has experienced 'running' of displacement piling in certain areas that are composed of uniform, loose sands. The Department has also experienced difficulty driving displacement piles in denser sands within cofferdams, as consecutive piles are driven, due to compaction of the in-situ sand during pile installation within the cofferdam footprint.

If boulders or cobbles are anticipated within the estimated length of the pile, consideration should be given to increasing the cast-in-place (CIP) pile shell thickness to reduce the potential of pile damage due to high driving stresses. Other alternatives are to investigate the use of pile points or the use of HP piles at the site.

Environmental factors may be significant in the selection of the pile type. Environmental factors include areas subject to high corrosion, bacterial corrosion, abrasion due to moving debris or ice, wave action, deterioration due to cyclic wetting and drying, strong current and gradual erosion of riverbed due to scour. Concrete piles are susceptible to corrosion when exposed to alkaline soil or strong chemicals, especially in rivers and streams. Steel piles can suffer serious electrolysis deterioration if placed in an environment near stray electrical currents. Cast-in-place concrete piling is generally the preferred pile type on structure widenings where displacement piles are required. Timber pile is not to be used, even if timber pile was used on the original structure.

Displacement pile consisting of tapered steel is proprietary and can be an efficient type of friction pile for bearing in loose to medium-dense granular soil. Tapered friction piles may need to be installed with the aid of water jetting in dense granular soil. Straight-sided friction piles are recommended for placement in cohesive soils underlain by a granular stratum to develop the greatest combined shaft and point resistance. Steel HP or open-end pipe piles are best suited for driving through obstructions or fairly competent layers to bedrock. Foundations such as pier bents which may be subject to large lateral forces when located in deep and/or swiftly moving water require piles that can sustain large bending forces. Precast, prestressed concrete pile is best suited for high lateral loading conditions but is seldom used on Wisconsin transportation projects.

11.3.1.12.1 Timber Piles

Current design practice is not to use timber piles.

11.3.1.12.2 Concrete Piles

The three principal types of concrete pile are cast-in-place (CIP), precast reinforced and prestressed reinforced. CIP concrete pile types include piles cast in driven steel shells that remain in-place, and piles cast in unlined drilled holes or shafts. Driven-type concrete pile is discussed below in this section. Concrete pile cast in drilled holes is discussed later in this chapter and include drilled shafts (11.3.2), micropiles (11.3.3), and augered cast-in-place piles (11.3.4).

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

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

11.3.1.12.2.1 Driven Cast-In-Place Concrete Piles

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

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

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

For consistency with WisDOT design practice, the steel shell is ignored when computing the axial structural resistance of driven CIP concrete pile that is symmetrical about both principal axes. This nominal (ultimate) axial structural resistance capacity is computed using the following equation, neglecting the contribution of the steel shell to resist compression: **LRFD [Equation 5.7.4.4-3]**.

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

Where:

$$P_n = 0.80(0.85 f'_c (A_g - A_{st})) + f_y A_{st}$$

Where:

P_u	=	Factored axial force effect (kips)
P_r	=	Factored axial resistance without flexure (kips)
ϕ	=	Resistance factor
P_n	=	Nominal axial resistance without flexure (kips)
A_g	=	Gross area of concrete pile section (inches^2)
A_{st}	=	Total area of longitudinal reinforcement (inches^2)
f_y	=	Specified yield strength of reinforcement (ksi)
f'_c	=	Concrete compressive strength (ksi)

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

$$P_n = 0.68 f'_c A_g$$

A resistance factor, ϕ, of 0.75 is used to compute the factored structural axial resistance capacity, as specified in **LRFD [5.5.4.2.1]**. For CIP piling there are no reinforcing ties, however the steel shell acts to confine concrete similar to ties.

$$P_r = 0.51 f_c A_g$$

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

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

A concrete compressive strength of 4 ksi is the minimum value required by specification, while a value of 3.5 ksi is used in the structural design computations. Pile capacities are maximums, based on an assumed concrete compressive strength of 3.5 ksi. The concrete compressive strength of 3.5 ksi is based on construction difficulties and unknowns of placement. The Geotechnical Site Investigation Report must be used as a guide in determining the nominal geotechnical resistance for the pile.

Any structural strength contribution associated with the steel shell is neglected in driven CIP concrete pile design. Therefore, environmentally corrosive sites do not affect driven CIP concrete pile designs. An exception is that CIP should not be used for exposed pile bents in corrosive environments as shown in the *Facilities Development Manual,* Procedure 13-1-15.

Based on the above equation, current WisDOT practice is to design driven cast-in-place concrete piles for factored (ultimate structural) axial compression resistances as shown in Table 11.3-5. See 6.3.2.1 for the typical style of plan notes showing axial resistance as well as required driving resistance on plans. **If less than the maximum axial resistance is required by design, state only the required corresponding driving resistance on the plans.** The minimum shell thickness is 0.219 inches for straight steel tube and 0.1793 inches for fluted steel shells, unless otherwise noted in the Geotechnical Site Investigation Report and stated in the project plans. Exposed piling (e.g. open pile bents) should not be less than 12 inches in diameter.

When cobbles or other difficult driving conditions are present, the minimum wall thickness for steel shells of driven cast-in-place concrete piles should be increased to 0.25 inches or thicker to facilitate driving without damaging the pile. A drivability analysis should be completed in design, to determine the required wall thickness based on site conditions and an assumed driving equipment.

Driven cast-in-place concrete pile is generally the most favorable displacement pile type since inspection of the steel shell is possible prior to concrete placement and more reliable control of concrete placement is attainable.

11.3.1.12.2.2 Precast Concrete Piles

Precast concrete pile can be divided into two primary types – reinforced concrete piles and prestressed concrete piles. These piles have parallel or tapered sides and are usually of rectangular or round cross section. Since the piles are usually cast in a horizontal position, the round cross section is not common because of the difficulty involved in filling a horizontal cylindrical form. Because of the somewhat variable subsurface conditions in Wisconsin and the need for variable length piles, these piles are currently not used in Wisconsin.

11.3.1.12.3 Steel Piles

Steel pile generally consist of either H-pile or pipe pile types. Both open-end and closed-end pipe pile are used. Pipe piles may be left open or filled with concrete, and can also have a structural shape or reinforcement steel inserted into the concrete. Open-end pipe pile can be socketed into bedrock with preboring.

Steel pile is typically top driven at the pile butt. However, closed-end pipe pile can also be bottom driven with a mandrel. Mandrels are generally not used in Wisconsin.

Steel pile can be used in friction, point-bearing, a combination of both, or rock-socketed piles. One advantage of steel pile is the ease of splicing or cutting to accommodate differing final constructed lengths.

Steel pile should not be used for exposed pile bents in corrosive environments as show in the *Facilities Development Manual, Procedure 13.1.15*.

The nominal (ultimate) axial structural compressive resistance of steel piles is designed in accordance with **LRFD [10.7.3.13.1]** as either non-composite or composite sections. Composite sections include concrete-filled pipe pile and steel pile that is encased in concrete. The nominal structural compressive resistance for non-composite and composite steel pile is further specified in **LRFD [6.9.4 and 6.9.5]**, respectively. The effective length of horizontally unsupported steel pile is determined in accordance with **LRFD [10.7.3.13.4]**. Resistance factors for the structural compression limit state are specified in **LRFD [6.5.4.2]**.

WisDOT policy item:

Specify a yield strength of 50 ksi for steel H-piles. Although 50 ksi is specified, the structural pile design shall use a yield strength of 36 ksi. The specified yield strength of 50 ksi may be used when performing drivability analyses. For steel pipe piles, 35 ksi shall be used for pile design and drivability analyses.

11.3.1.12.3.1 H-Piles

Steel piles are generally used for point-bearing piles and typically employ what is known as the HP-section (often called H-piles for brevity). Steel H-piles are rolled sections with wide flanges such that the depth of the section and the width of the flanges are approximately equal. The cross-sectional area and volume displacement are relatively small and as a result, H-piles can be driven through compact granular materials and slightly into soft rock. Also, steel piles have little or no effect in causing ground swelling or raising of adjacent piles. Because of the small volume of H-piles, they are considered “non-displacement” piling.

H-piles are available in many sizes and lengths. Unspliced pile lengths up to 140 feet and spliced pile lengths up to 230 feet have been driven. Typical pile lengths range from 40 to 120 feet. Common H-pile sizes vary between 10 and 14 inches.

The current WisDOT practice is to design driven H-piles for the factored (ultimate structural) axial compression resistance as shown in Table 11.3-5. These values are based on $\phi_c = 0.5$ for severe driving conditions **LRFD [6.5.4.2]**. See 6.3.2.1 for the typical style of plan notes showing axial resistance as well as required driving resistance on plans. **If less than the maximum axial resistance is required by design, state only the required corresponding driving resistance on the plans.**

Since granular soil is largely incompressible, the principal action at the tip of the pile is lateral displacement of soil particles. Although it is an accepted fact that steel piles develop extremely high loads per pile when driven to point-bearing on rock, some misconceptions still remain that H-piles cannot function as friction piles. Load tests indicate that steel H-piles can function quite satisfactorily as friction piles in sand, sand-clay, silt-and-sand or hard clay. However, they are not as efficient as displacement piles in these conditions and typically drive to greater depths. 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.

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.

11.3.1.12.3.2 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, which are generally used for geotechnical design on WisDOT projects.

Condition/Resistance Determination Method			Resistance Factor
Static Analysis – Used in Design Phase	Nominal Resistance of Single Pile in Axial Compression, φ_{stat}	Skin Friction and End Bearing in Clay and Mixed Soil Alpha Method	0.35
		Skin Friction and End Bearing in Sand Nordlund/Thurman Method	0.45
		Point Bearing in Rock	0.45
	Block Failure, φ_{bl}	Clay	0.60
	Uplift Resistance of Single Pile, φ_{up}	Clay and Mixed Soil Alpha Method	0.25
		Sand Nordlund Method	0.35
	Horizontal Resistance of Single Pile or Pile Group	All Soil Types and Rock	1.0
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}		FHWA-modified Gates dynamic pile driving formula (end of drive condition only)	0.50
		Wave equation analysis, without pile dynamic measurements or load test, at end of drive condition only	0.50
		Driving criteria established by dynamic test with signal matching at beginning of redrive conditions only of at least one production pile per substructure, but no less than the number of tests per site provided in **LRFD [Table 10.5.5.2.3-3]**; quality control of remaining piles by calibrated wave equation and/or dynamic testing	0.65

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.

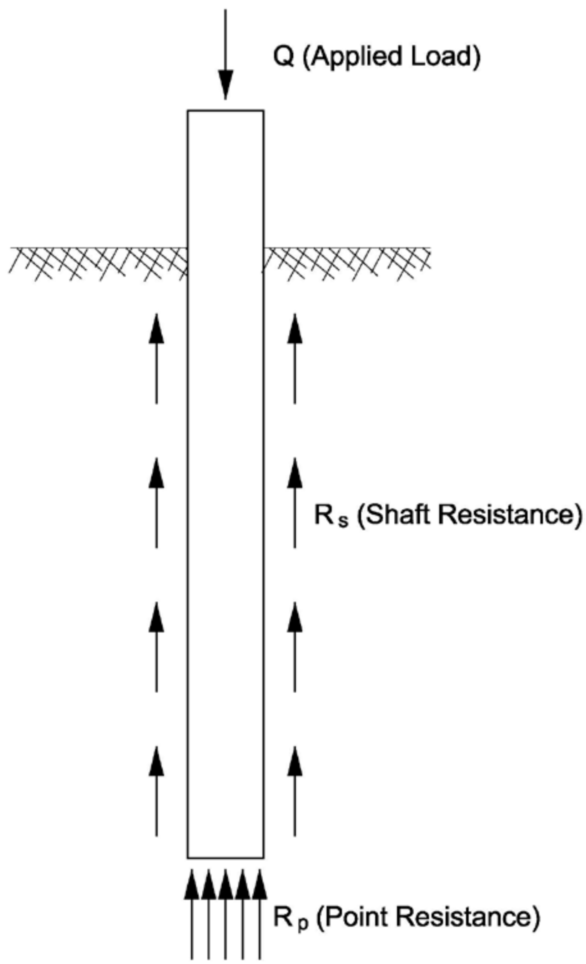


Figure 11.3-1
Resistance Distribution for Axially-Loaded Pile

11.3.1.15.1 Shaft Resistance

The shaft resistance of a pile is estimated by summing the frictional resistance developed in each of the different soil strata.

For non-cohesive (granular) soil, the total shaft resistance can be calculated using the following equation (based on the Nordlund/Thurman Method):

$$R_s = \Sigma C_d D K_\delta C_F \sigma_v' \frac{\sin(\delta + \omega)}{\cos(\omega)}$$

Where:

R_s	=	Total shaft resistance capacity (tons)
C_d	=	Pile perimeter (feet)
D	=	Pile segment length (feet)
K_δ	=	Coefficient of lateral earth pressure at mid-point of soil layer under consideration from **LRFD [Figures 10.7.3.8.6f-1 through 10.7.3.8.6f-4]**
C_F	=	Correction factor for K_δ when $\delta \neq \phi_f$, from **LRFD [Figure 10.7.3.8.6f-5],** whereby ϕ_f = angle of internal friction for drained soil
σ_v'	=	Effective overburden pressure at midpoint of soil layer under consideration (tsf)
δ	=	Friction angle between the pile and soil obtained from **LRFD [Figure 10.7.3.8.6f-6]** (degrees)
ω	=	Angle of pile taper from vertical (degrees)

For cohesive (fine-grained) soil, the total shaft resistance can be calculated using the following equation (based on the alpha method):

$$R_s = \Sigma \alpha S_u C_d D$$

Where:

R_s	=	Total (nominal) shaft resistance capacity (tons)
α	=	Adhesion factor based on the undrained shear strength from **LRFD [Figure 10.7.3.8.6b-1]**
S_u	=	Undrained shear strength (tsf)
C_d	=	Pile perimeter (feet)
D	=	Pile segment length (feet)

Typical values of nominal shaft resistance for various soils are presented in Table 11.3-2 and Table 11.3-3. The values presented are average ranges and are intended to provide orders of magnitude only. Other conditions such as layering sequences, drilling information, ground water, thixotropy and clay sensitivity must be evaluated by experienced geotechnical engineers and analyzed using principles of soil mechanics.

Soil Type	q_u (1) (tsf)	Nominal Shaft Resistance (psf)
Very soft clay	0 to 0.25	---
Soft clay	0.25 to 0.5	200 to 450
Medium clay	0.5 to 1.0	450 to 800
Stiff clay	1.0 to 2.0	800 to 1,500
Very stiff clay	2.0 to 4.0	1,500 to 2,500
Hard clay	4.0	2,500 to 3,500
Silt	---	100 to 400
Silty clay	---	400 to 700
Sandy clay	---	400 to 700
Sandy silt	---	600 to 1,000
Dense silty clay	---	900 to 1,500

(1) Unconfined Compression Strength

Table 11.3-2
Typical Nominal Shaft Resistance Values for Cohesive Material

Soil Type	N_{160} (1)	Nominal Shaft Resistance (psf)
Very loose sand and silt or clay	0 to 6	50 to 150
Medium sand and silt or clay	6 to 30	400 to 600
Dense sand and silt or clay	30 to 50	600 to 800
Very dense sand and silt or clay	over 50	800 to 1,000
Very loose sand	0 to 4	700 to 1,700
Loose sand	4 to 10	700 to 1,700
Firm sand	10 to 30	700 to 1,700
Dense sand	30 to 50	700 to 1,700
Very dense sand	over 50	700 to 1,700
Sand and gravel	---	1,000 to 3,000
Gravel	---	1,500 to 3,500

(1) Standard Penetration Value (AASHTO T206) corrected for both overburden and hammer efficiency effects (blows per foot).

Table 11.3-3
Typical Nominal Shaft Resistance Values for Granular Material

Shaft resistance values are dependent upon soil texture, overburden pressure and soil cohesion but tend to increase with depth. However, experience in Wisconsin has shown that shaft resistance values in non-cohesive materials reach constant final values at depths of 15 to 25 pile diameters in loose sands and 25 to 35 pile diameters in firm sands.

In computing shaft resistance, the method of installation must be considered as well as the soil type. The method of installation significantly affects the degree of soil disturbance, the lateral stress acting on the pile, the friction angle and the area of contact. Shafts of prebored piles do not always fully contact the soil; therefore, the effective contact area is less than the shaft surface area. Driving a pile in granular material densifies the soil and increases the friction angle. Driving also displaces the soil laterally and increases the horizontal stress acting on the pile. Disturbance of clay soil from driving can break down soil structure and increase pore pressures, which greatly decreases soil strength. However, some or all of the strength recovers following reconsolidation of the soil due to a decrease in excess pore pressure over time. Use the initial soil strength values for design purposes. The type and shape of a pile also affects the amount of shaft resistance developed, as described in 11.3.1.12.

11.3.1.15.2 Point Resistance

The point resistance, or end bearing capacity, of a pile is estimated from modifications to the bearing capacity formulas developed for shallow footings.

For non-cohesive soils, point resistance can be calculated using the following equation (based on the Nordlund/Thurman Method):

$$R_p = A_p \, \alpha_t \, N'_q \, \sigma_v' \leq q_L \, A_p$$

Where:

R_p	=	Point resistance capacity (tons)
A_p	=	Pile end area (feet^2)
α_t	=	Dimensionless factor dependent on depth-width relationship from **LRFD [Figure 10.7.3.8.6f-7]**
N'_q	=	Bearing capacity factor from **LRFD [Figure 10.7.3.8.6f-8]**
σ_v'	=	Effective overburden pressure at the pile point $\leq$ 1.6 (tsf)
q_L	=	Limiting unit point resistance from **LRFD [Figure 10.7.3.8.6f-9]** (tsf)

For cohesive soils, point resistance can be calculated using the following equation:

$$R_p = 9 S_u A_p$$

Where:

R_p	=	Point resistance capacity (tons)
S_u	=	Undrained shear strength of the cohesive soil near the pile base (tsf)
A_p	=	Pile end area (feet^2)

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

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

11.3.1.15.3 Group Capacity

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

11.3.1.16 Lateral Load Resistance

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

- Pile group configuration.
- Pile stiffness.

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

If a more detailed lateral load investigation is desired, a P-y analysis is typically performed using commercially available software such as COM624P, FB Multi-Pier or L-Pile. A resistance factor of 1.0 is applied to the soil response when performing a P-y analysis using factored loads since the soil response represents a nominal (ultimate) condition. For a more detailed analysis of lateral loads and displacements, refer to the listed FHWA design references at the end of this chapter or a geotechnical engineering book.

WisDOT policy item:

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

11.3.1.17 Other Design Considerations

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

11.3.1.17.1 Downdrag Load

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

The factored axial compression resistance values given for H-piles in Table 11.3-5 are conservative and based on Departmental experience to avoid overstressing during driving. For H-piles in end bearing, loading from downdrag is allowed in addition to the normal pile loading, since this is a post-driving load. Use the values given in Table 11.3-5 and design piling as usual. Additionally, up to 45, 60, and 105 tons downdrag for HP 10x42, HP 12x53, and HP 14x73 piles respectively is allowed when the required driving resistance is determined by the modified Gates dynamic 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 equation techniques. These techniques are used to document that the assumed pile driving hammers are capable of mobilizing the required nominal (ultimate) resistance of the pile at driving stress levels less than the factored driving resistance of the pile. Drivability can often be the controlling strength limit state check for a pile foundation. This is especially true for high capacity piles driven to refusal on rock.

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

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

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

The following presents potential sources of wave equation errors.

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

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

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

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

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

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

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

Soil Type	Recommended Soil Set Up Factor[1]	Percentage Loss of Soil Strength during Driving
Clay	2.0	50 percent
Silt – Clay	1.5[2]	33 percent
Silt	1.5	33 percent
Sand – Clay	1.5	33 percent
Sand – Silt	1.2	17 percent
Fine Sand	1.2	17 percent
Sand	1.0	0 percent
Sand - Gravel	1.0	0 percent

Notes:

1. Confirmation with local experience recommended
2. The value of 1.5 is higher than the FHWA Table 9-19 value of 1.0 based upon WisDOT experience.

Table 11.3-4
Soil Resistance Factors

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

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

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

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

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

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

11.3.1.17.6 Scour

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

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

Where:

R_n	=	Nominal shaft resistance capacity, adjusted for scour effect (tons)
$R_{n\text{-stat}}$	=	Nominal shaft resistance based on static analysis, without scour consideration (tons)
$R_{n\text{-scour}}$	=	Nominal shaft resistance lost (negative value) over the exposed pile length due to scour (tons)

WisDOT policy item:

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

11.3.1.17.7 Typical Pile Resistance Values

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

Pile Size	Shell Thickness (inches)	Concrete or Steel Area (A_g or A_s) (in^2)	Nominal Axial Compression Resistance (Pn) (tons) (2)(3)(6)	Resistance Factor (ϕ)	Factored Axial Compression Resistance (Pr) (tons) (4)	Resistance Factor φ_{dyn}	Required Driving Resistance (Rn_{dyn}) (tons) (5)
Cast in Place Piles							
10 ¾“	0.219	83.5	99.4	0.75	55(8)	0.5	110
10 ¾“	0.250	82.5	98.2	0.75	65(8)	0.5	130
10 ¾“	0.365	78.9	93.8	0.75	75	0.5	150
10 ¾“	0.500	74.7	88.8	0.75	75(9)	0.5	150
12 ¾”	0.250	118.0	140.4	0.75	80(8)	0.5	160
12 ¾”	0.375	113.1	134.6	0.75	105	0.5	210
12 ¾”	0.500	108.4	129.0	0.75	105(9)	0.5	210
14”	0.250	143.1	170.3	0.75	85(8)	0.5	170
14”	0.375	137.9	164.1	0.75	120	0.5	240
14”	0.500	132.7	158.0	0.75	120(9)	0.5	240
16”	0.375	182.6	217.3	0.75	145(8)	0.5	290
16”	0.500	176.7	210.3	0.75	160	0.5	320
H-Piles							
10 x 42	NA(1)	12.4	310.0	0.50	90(10)	0.5	180
12 x 53	NA(1)	15.5	387.5	0.50	110(10)	0.5	220
14 x 73	NA(1)	21.4	535.0	0.50	125(10)	0.5	250

Table 11.3-5
Typical Pile Resistance Values

Notes

1. NA – not applicable

2. For CIP Piles: Pn = 0.8 (0.85 * f’c * Ag + fy * As) **LRFD [5.7.4.4-3]**. Neglecting the steel shell, equation reduces to 0.68 * f’c * Ag.

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

3. For H-Piles: Pn = (0.66^{λ} * Fe * As) **LRFD [6.9.5.1-1]** (λ = 0 for piles embedded in the ground below the substructure, i.e. no unsupported lengths)

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

4. Pr = ϕ * Pn

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

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

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

   - Rn_{dyn} = Pr / φ_{dyn}

     φ_{dyn} = 0.5 for construction driving criteria using modified Gates dynamic formula)

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

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

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

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

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

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

11.3.1.18 Construction Considerations

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

11.3.1.18.1 Pile Hammers

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

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

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

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

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

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

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

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

11.3.1.18.2 Driving Formulas

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

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

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

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

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

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

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

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

Where:

R_R = Factored pile resistance (tons)

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

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

E_d	=	Energy delivered by the hammer per blow (lb-foot)
s	=	Average penetration in inches per blow for the final 10 blows (inches/blow)

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

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

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

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

11.3.1.18.3 Field Testing

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

- For test driving, to determine the length of pile required prior to placing purchasing orders.

- For load testing, to verify actual pile capacity versus design capacity for nominal axial resistance.

11.3.1.18.3.1 Installation of Test Piles

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

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

11.3.1.18.3.2 Static Pile Load Tests

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

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

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

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

- The other piles are of the same type, material and size as the test piles.

- Subsoil conditions are comparable to those at the test pile locations.

- Installation methods and equipment used are the same as, or comparable to, those used for the test piles.

- Piles are driven to the same penetration depth or resistance or both as the test piles to compensate for variations in the vertical position and density of the bearing strata.

11.3.1.19 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. From a practical point of view the resistance factor for a deep foundation is the relationship between the Factored Axial Compression Resistance (FACR) and the Required Driving Resistance (RDR). The potential resistance factors (see Table 11.3-1) for use in deep foundation design are as follows:

Methods Used to Determine Required Driving Resistance	Resistance Factor
FHWA-modified Gates dynamic pile driving formula (end of drive condition only).	0.50
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 one production pile per substructure, but no less than the number of tests per site provided in LRFD[Table 10.5.5.2.3-3]. Quality control of remaining piles by calibrated wave equation and/or dynamic testing.	0.65
Static Pile Load Test(s) and dynamic test (PDA) with signal matching (CAPWAP) at beginning of redrive conditions only, of at least one production pile per substructure, but no less than the number of tests per site provided in LRFD[Table 10.5.5.2.3-3]. Quality control of remaining piles by calibrated wave equation and/or dynamic testing.	0.80

Table 11.3-6
Resistance Factors and Deep Foundation Methods of Construction Monitoring

The typical method for a majority of the Department's deep foundation substructures is using the modified Gates dynamic formula to determine the RDR and to use a resistance factor of 0.50. 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 assume the use of the Department's PDA and the Department's personnel to collect the required data and illustrate the potential cost savings/expenses. If the consultant's PDA and/or the consultant's personnel are used to collect the required data then the cost savings will be significantly reduced:

Pier
Pier Example: 12 x 53 H-piles to an estimated length of 100 feet at a unit cost of $50/foot.
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 (**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). Total Cost = 32 piles x 100 feet x $50/ft = $160,000
PDA/CAPWAP: RDR = 220 tons, FACR = 143 tons, Load on Pier = 3,500 tons, Number of Piles = 3,500 tons / 143 tons = 25 piles, 3 tests at end of initial driving (EOID), 3 tests at beginning of restrike (BOR) Total Cost =Number of Piles x Estimated Length of Pile x Cost/Linear Foot of Pile + ((CAPWAP cost ($1,000/test) + contractor cost ($100/pile) + DOT cost ($125/pile)) Total Cost = 25 piles x 100 feet x $50/ft + ((6 tests x $1,000/test) + (6 piles x $100/pile) + (6 piles x $125/pile)) = $132,1350
PDA/CAPWAP Savings = $27,650/pier
Abutment
Abutment Example: 12 x 53 H-piles to an estimated length of 40 feet at a unit cost of $50/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 40 feet x $50/ft = $18,000
PDA/CAPWAP: RDR = 220 tons, FACR = 143 tons, Load on Abut = 980 tons, Number of Piles = 980 tons / 140 tons = 7 piles, however because of maximum spacing requirements the design will need 8 piles, 1 test at end of initial driving (EOID), 1 test at beginning of restrike (BOR) Total Cost =Number of Piles x Estimated Length of Pile x Cost/Linear Foot of Pile +

((CAPWAP cost ($1,000/test) + contractor cost ($100/pile) + DOT cost ($125/pile)) Total Cost = 8 piles x 40 feet x $50/ft + ((2 test x $1,000/test) + (2 piles x $100/pile) + (2 piles x $125/pile)) = $18,450
PDA/CAPWAP Cost = $450/abutment

Table 11.3-7

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 11.3-8 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 11.3-8

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

For drilled shafts, the base geotechnical resistance factors in Table 11.3-8 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 11.3-8 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 11.3-8 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 11.3-8 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]**

11.3.2.3.1 Point Resistance

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

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

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

11.3.2.3.2 Group Capacity

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

11.3.2.4 Lateral Load Resistance

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

11.3.2.5 Other Considerations

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

11.3.3 Micropiles

11.3.3.1 General

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

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

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

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

11.3.3.2 Design Guidance

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

11.3.4 Augered Cast-In-Place Piles

11.3.4.1 General

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

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

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

11.3.4.2 Design Guidance

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

11.4 References

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11.5 Design Examples

WisDOT will provide design examples.

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

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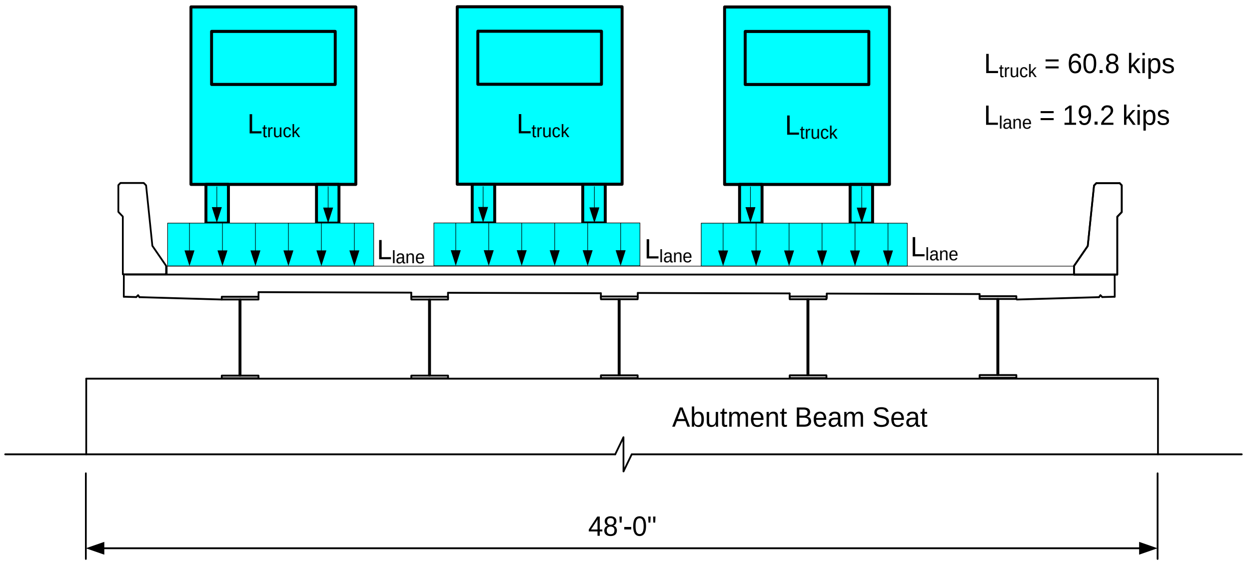

Figure 12.8-3
Live Load on Abutment Beam Seat

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

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

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

12.8.2 Load Modifiers and Load Factors

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

Description	Load Modifier
Ductility	1.00
Redundancy	1.00
Operational classification	1.00

Table 12.8-1
Load Modifiers Used in Abutment Design

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

Direction of Load	Specific Loading	Load Factor		
		Strength I		Service I
		Max.	Min.	
Load factors for vertical loads	Superstructure DC dead load	1.25	0.90	1.00
	Superstructure DW dead load	1.50	0.65	1.00
	Superstructure live load	1.75	1.75	1.00
	Approach slab dead load	1.25	0.90	1.00
	Approach slab live load	1.75	1.75	1.00
	Wheel loads located directly on the abutment backwall	1.75	1.75	1.00
	Earth surcharge	1.50	0.75	1.00
	Earth pressure	1.35	1.00	1.00
	Water load	1.00	1.00	1.00
	Live load surcharge	1.75	1.75	1.00
Load factors for horizontal loads	Substructure wind load, WS	0.00	0.00	0.00
	Superstructure wind load, WS	0.00	0.00	0.30
	Superstructure wind on LL, WL	0.00	0.00	1.00
	Vehicular braking force from live load	1.75	1.75	1.00
	Temperature and shrinkage*	1.20*	0.50*	1.00
	Earth pressure (active)	1.50	0.90	1.00
	Earth surcharge	1.50	0.75	1.00
	Live load surcharge	1.75	1.75	1.00

Table 12.8-2
Load Factors Used in Abutment Design

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

12.8.3 Live Load Surcharge

The equivalent heights of soil for vehicular loading on abutments perpendicular to traffic are as presented in **LRFD [Table 3.11.6.4-1]** and in Table 12.8-3. Values are presented for various abutment heights. The abutment height, as used in Table 12.8-3, is taken as the distance between the top surface of the backfill at the back face of the abutment and the bottom of the footing along the pressure surface being considered. Linear interpolation

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13.1 General

Piers are an integral part of the load path between the superstructure and the foundation. Piers are designed to resist the vertical loads from the superstructure, as well as the horizontal superstructure loads not resisted by the abutments. The magnitude of the superstructure loads applied to each pier shall consider the configuration of the fixed and expansion bearings, the bearing types and the relative stiffness of all of the piers. The analysis to determine the horizontal loads applied at each pier must consider the entire system of piers and abutments and not just the individual pier. The piers shall also resist loads applied directly to them, such as wind loads, ice loads, water pressures and vehicle impact.

Bridges being designed with staged construction, whether new or rehabilitation, shall satisfy the requirements of LRFD (or LFD, if applicable) for each construction stage. Utilize the same load factors, resistance factors, load combinations, etc. as required for the final configuration, unless approved by Chief Structures Development Engineer at WisDOT.

WisDOT policy item:

At this time, evaluation and plan preparations for accommodating a noted allowance for a precast pier option as indicated in 7.1.4.1.2 is only required for I-39/90 Project bridges. All other locations statewide may consider providing a noted allowance for a precast option. Contact the Bureau of Structures Development Section for further guidance.

13.1.1 Pier Type and Configuration

Many factors are considered when selecting a pier type and configuration. The engineer should consider the superstructure type, the characteristics of the feature crossed, span lengths, bridge width, bearing type and width, skew, required vertical and horizontal clearance, required pier height, aesthetics and economy. For bridges over waterways, the pier location relative to the floodplain and scour sensitive regions shall also be considered.

The connection between the pier and superstructure is usually a fixed or expansion bearing which allows rotation in the longitudinal direction of the superstructure. This has the effect of eliminating longitudinal moment transfer between the superstructure and the pier. In rare cases when the pier is integral with the superstructure, this longitudinal rotation is restrained and moment transfer between the superstructure and the pier occurs. Pier types illustrated in the Standard Details shall be considered to be a pinned connection to the superstructure.

On grades greater than 2 percent, the superstructure tends to move downhill towards the abutment. The low end abutment should be designed as fixed and the expansion joint or joints placed on the uphill side or high end abutment. Consideration should also be given to fixing more piers than a typical bridge on a flat grade.

13.1.2 Bottom of Footing Elevation

The bottom of footing elevation for piers outside of the floodplain is to be a minimum of 4' below finished ground line unless the footings are founded on solid rock. This requirement is intended to place the bottom of the footing below the frost line.

A minimum thickness of 2'-0" shall be used for spread footings and 2'-6" for pile-supported footings. Spread footings are permitted in streams only if they are founded on rock. Pile cap footings are allowed above the ultimate scour depth elevation if the piling is designed assuming the full scour depth condition.

The bottom of footing elevation for pile cap footings in the floodplain is to be a minimum of 6' below stable streambed elevation. Stable streambed elevation is the normal low streambed elevation at a given pier location when not under scour conditions. When a pile cap footing in the floodplain is placed on a concrete seal, the bottom of footing is to be a minimum of 4' below stable streambed elevation. The bottom of concrete seal elevation is to be a minimum of 8' below stable streambed elevation. These requirements are intended to guard against the effects of scour.

13.1.3 Pier Construction

Except for pile encased piers (see Standard for Pile Encased Pier) and seal concrete for footings, all footing and pier concrete shall be placed in the dry. Successful underwater concreting requires special concrete mixes, additives and placement procedures, and the risk of error is high. A major concern in underwater concreting is that the water in which the concrete is placed will wash away cement and sand, or mix with the concrete, and increase the water-to-cement ratio. It was previously believed that if the lower end of the tremie is kept immersed in concrete during a placement, then the new concrete flows under and is protected by previously placed concrete. However, tests performed at the University of California at Berkeley show that concrete exiting a tremie pipe may exhibit many different flow patterns exposing more concrete to water than expected. A layer of soft, weak and water-laden mortar called laitance may also form within the pour. Slump tests do not measure shear resistance, which is the best predictor of how concrete will flow after exiting a tremie pipe.

Footing excavation adjacent to railroad tracks which falls within the critical zone shown on Standard for Highway Over Railroad Design Requirements requires an approved shoring system. Excavation, shoring and cofferdam costs shall be considered when evaluating estimated costs for pier construction, where applicable. Erosion protection is required for all excavations.

Pile bents may only be specified where the structure is located within Area 3, as shown in the *Facilities Development Manual 13-1-15*, *Attachment 15.1* and where the piles are not exposed to water with characteristics that are likely to cause accelerated corrosion.

The minimum cap size shall be 3' wide by 3'-6" deep and the piles shall be embedded into the cap a minimum of 2'-0.

13.2.3 Pile Encased Piers

Pile encased piers are similar to pile bents except that a concrete encasement wall surrounds the piles. They are most commonly used for small to intermediate stream crossings where a pile bent pier is not feasible. Pile encased piers are detailed on Standard for Pile Encased Pier.

An advantage of this pier type is that the concrete encasement wall provides greater resistance to lateral forces than a pile bent. Also the hydraulic characteristics of this pier type are superior to pile bents, resulting in a smoother flow and reducing the susceptibility of the pier to scour at high water velocities. Another advantage is that floating debris and ice are less likely to accumulate against a pile encased pier than between the piles of a pile bent. Debris and ice accumulation are detrimental because of the increased stream force they induce. In addition, debris and ice accumulation cause turbulence at the pile, which can have the effect of increasing the local scour potential.

Pile sections shall be limited to 10", 12" or 14" steel HP piles, or 10¾", 12¾" or 14" diameter cast-in-place concrete piles with steel shells. Minimum center-to-center spacing is 3'. Where difficult driving conditions are expected, oil field pipe may be specified in the design. A minimum of five piles per pier shall be used. When a satisfactory design cannot be developed with one of these pile sections at the minimum spacing, another pier type should be selected. The inside and outside piles shall be driven vertically.

Pile encased piers should not be used for normal water depths greater than 10', since this is the maximum practical depth for setting formwork and placing the reinforcing steel. Total pier height shall be less than 25 feet.

All bearings supporting a superstructure utilizing pile encased piers shall be fixed bearings or semi-expansion.

The connection between the superstructure and the pier shall be designed to transmit the portion of the superstructure design loads assumed to be taken by the pier.

The concrete wall shall be a minimum of 2'-6" thick. The top 3' of the wall is made wider if a larger bearing area is required. See Standard for Pile Encased Pier for details. The bottom of the wall shall be placed 2' to 4' below stable streambed elevation, depending upon stream velocities and frost depth.

The concrete in the encasement wall may be placed under water if the procedure detailed in 13.14 is followed.

13.2.4 Solid Single Shaft / Hammerheads

Solid single shaft piers are used for all types of crossings and are detailed on Standards for Hammerhead Pier and for Hammerhead Pier – Type 2. The choice between using a multi-column pier and a solid single shaft pier is based on economics and aesthetics. For high level bridges, a solid single shaft pier is generally the most economical and attractive pier type available.

The massiveness of this pier type provides a large lateral load capacity to resist the somewhat unpredictable forces from floating ice, debris and expanding ice. They are suitable for use on major rivers adjacent to shipping channels without additional pier protection. When used adjacent to railroad tracks, crash walls are not required.

If a cofferdam is required and the upper portion of a single shaft pier extends over the cofferdam, an optional construction joint is provided 2’ above the normal water elevation. Since the cofferdam sheet piling is removed by extracting vertically, any overhead obstruction prevents removal and this optional construction joint allows the contractor to remove sheet piling before proceeding with construction of the overhanging portions of the pier.

A hammerhead pier shall not be used when the junction between the cap and the shaft would be less than the cap depth above normal water. Hammerhead piers are not considered aesthetically pleasing when the shaft exposure above water is not significant. A feasible alternative in this situation would be a wall type solid single shaft pier or a multi-column pier. On a wall type pier, both the sides and ends may be sloped if desired, and either a round, square or angled end treatment is acceptable. If placed in a waterway, a square end type is less desirable than a round or angled end.

13.2.5 Aesthetics

Refer to 13.13 for suggested alternative pier shapes. These shapes are currently being studied so no standard details are shown. It is desirable to standardize alternate shapes for efficiency and economy of construction. Use of these alternate pier shapes for aesthetics should be approved by the Chief Structures Development Engineer so that standard details can be developed.

Refer to Chapter 4 for additional information about aesthetics.

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	0.50	0.00
Strength III	1.25	0.90	1.50	0.65	0.00	1.00	1.40	0.00	1.00	0.50	0.00
Strength V	1.25	0.90	1.50	0.65	1.35	1.00	0.40	1.00	1.00	0.50	0.00
Service I	1.00	1.00	1.00	1.00	1.00	1.00	0.30	1.00	1.00	1.00	0.00
Extreme Event II	1.25	0.90	1.50	0.65	0.50	1.00	0.00	0.00	1.00	0.00	1.00

Table 13.5-1
Load Factors

13.5.2 Expansion Piers

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

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

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

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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)*** 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.

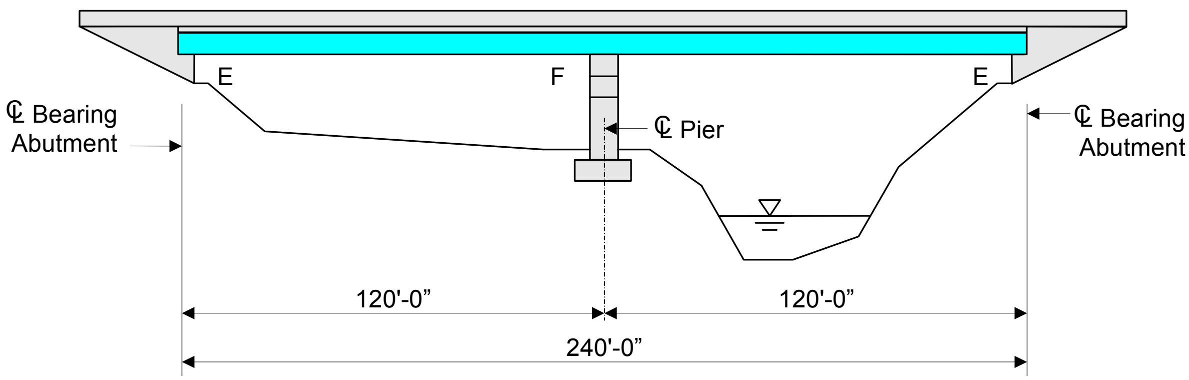

Figure E13-1.1-1
Bridge Elevation

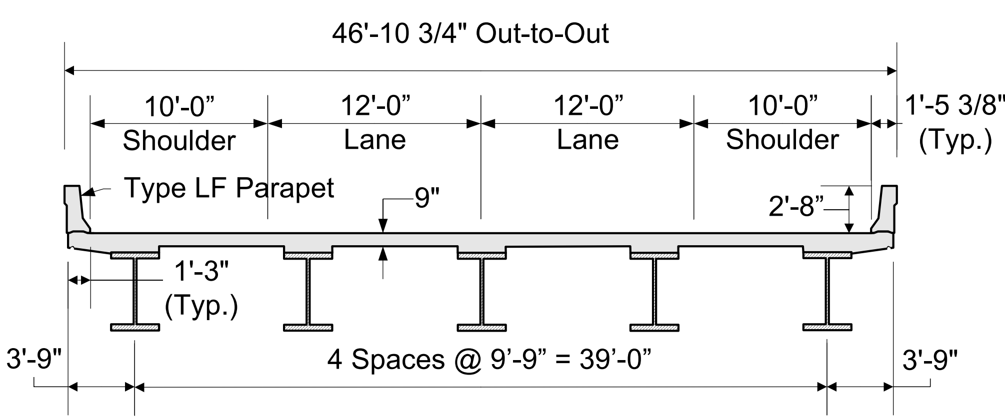

Figure E13-1.1-2
Bridge Cross Section

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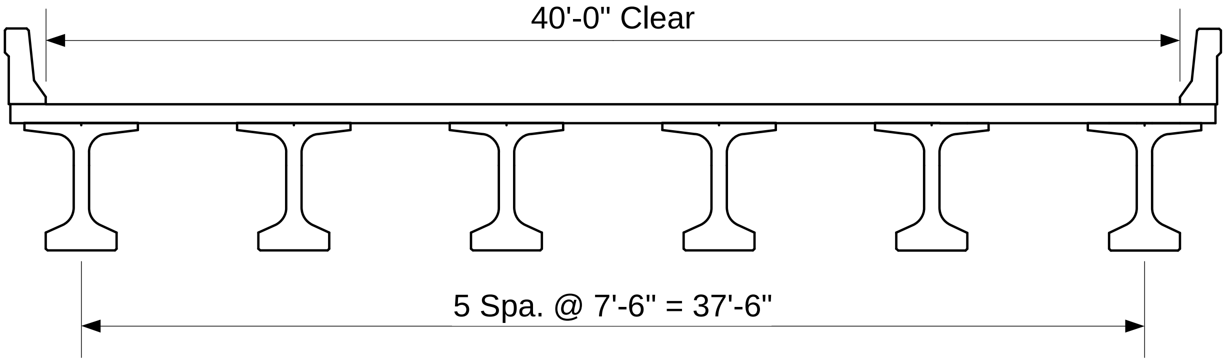
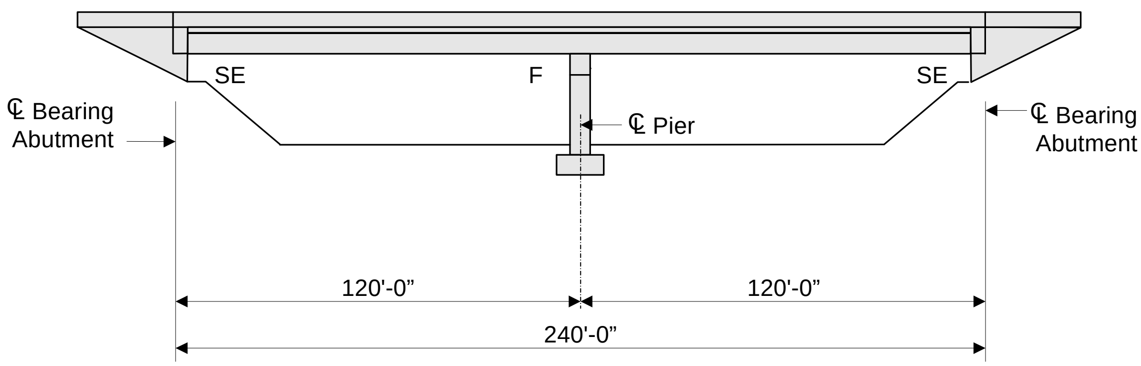
E13-2 Multi-Column Pier Design Example - LRFD

2 Span Bridge, 54W, LRFD Design



This pier is designed for the superstructure as detailed in example **E19-2**. This is a two-span prestressed girder grade separation structure. Semi-expansion bearings are located at the abutments, and fixed bearings are used at the pier.



E13-2.1 Obtain Design Criteria

This multi-column pier design example is based on ***AASHTO LRFD Bridge Design Specifications, (Seventh Edition).*** The design methods presented throughout the example are meant to be the most widely used in general bridge engineering practice. Calculations are only shown for the pier cap. For example column and footing calculations, see example E13-1.

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.

E13-2.1.1 Material Properties:

$w_c := 0.150$ Concrete density, kcf

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14.1 Introduction

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

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

WisDOT policy item:

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

14.1.1 Wall Development Process

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

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

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

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

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

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

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

14.1.1.1 Wall Numbering System

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

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

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

cost effective ranges. Surface conditions must also be considered. For instance, if soft compressible soils are present, walls that can tolerate larger settlements and movements must be considered. MSE walls are generally more economical for fill locations than CIP cantilever walls.

Cut/fill Walls

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

14.3.1.3 Site Characteristics

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

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

14.3.1.4 Miscellaneous Design Considerations

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

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

14.3.1.5 Right of Way Considerations

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

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

14.3.1.6 Utilities and Other Conflicts

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

14.3.1.7 Aesthetics

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

14.3.1.8 Constructability Considerations

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

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

14.3.1.9 Environmental Considerations

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

14.3.1.10 Cost

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

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

The non-metallic or extensible reinforcement includes the following:

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

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

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

14.6.2.3 Facing Elements

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

Major facing types are:

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

Segmental Precast Concrete Panels

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

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

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

Concrete Modular Blocks Facings

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

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

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

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

front of the wall could occur during the service life of the structure and lead to partial or complete loss of passive resistance.

Interface sliding resistance between concrete blocks shall be calculated using the corrected wall weight based on the calculated hinge height in accordance with **LRFD Figure [11.10.6.4.4b-1]**. Interface friction resistance parameters shall be based on NCMA method. Shear between the blocks must be resisted by friction, keys or pins.

14.7.1.2.3 Bearing Resistance

The bearing resistance of the walls shall be computed in accordance with **LRFD [10.6.3.1]**.

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

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

$q_r = \phi_b q_n \geq \sigma_v$

Where:

q_n	=	Nominal bearing resistance computed using **LRFD [10.6.3.1.2-a]**
$\sum V$	=	Summation of Vertical loads
B	=	Base width
e	=	Eccentricity
ϕ_b	=	0.55 **LRFD Table [11.5.6-1]**

14.7.1.2.4 Eccentricity Check

The eccentricity check shall be performed in accordance with **LRFD [11.11.4.4].** The location of the resultant force should be within the middle half of the base width (e< B/4) for footings on soil, and within (3B/8) for footings on rock.

14.7.1.3 Settlement

The vertical and lateral displacements of prefabricated modular retaining walls must be evaluated for all applicable dead and live load combinations at Service I limit states using procedures described in14.4.7.2 and compared with tolerable movement criteria presented in 14.4.7.2.1. In general, lateral movements of walls on shallow foundations can be estimated assuming the wall rotates or translates as a rigid body due to the effects of earth loads and differential settlements along the base of the wall.

14.7.1.4 Overall Stability

The overall (global) stability shall be evaluated in accordance with **LRFD [11.6.2.3]** and in accordance with 14.4.7.3, with the exception that the entire mass of the modular walls (or the "foundation load"), may be assumed to contribute to the overall stability of the slope. The overall stability check shall be performed by the Geotechnical Engineering Unit or Consultant of record.

14.7.1.5 Summary of Design Requirements

1. Stability Evaluations
   - External Stability
     - Eccentricity Check
     - Bearing Check
     - Sliding
   - Settlement
   - Overall/Global
2. Block Data
   - One piece block
   - Minimum thickness of front face = 4 inches
   - Minimum thickness of internal cavity walls other than front face = 2 inches
   - 28 day concrete strength = 5000 psi
   - Maximum water absorption rate by weight = 5%
3. Traffic Surcharge
   - Traffic live load surcharge = 240 lb/ft^2
   - If no traffic live load is present, use 100 lb/ft^2 live load for construction equipment
4. Retained Soil
   - Unit weight γ_f = 120 lb/ft^3
   - Angle of internal friction as determined by Geotechnical Engineer

14.14 Contract Plan Requirements

The following minimum information shall be required on the plans.

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

14.15 Construction Documents

14.15.1 Bid Items and Method of Measurement

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

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

Non-proprietary retaining walls are bid based on the quantity of materials used to construct the wall such as concrete, reinforcing steel, piling, etc. These walls are:

- Cast-in-Place Concrete Cantilever Walls
- Post-and-Panel Walls
- Steel Sheet Piling Walls

14.15.2 Special Provisions

The Structures Design Section has Standard Special Provisions for:

- Wall Modular Block Gravity LRFD, Item SPV.0165.
- Wall Modular Block Mechanically Stabilized Earth LRFD, Item SPV.0165.
- Wall Concrete Panel Mechanically Stabilized Earth LRFD, Item SPV.0165.
- Wall Concrete Panel Mechanically Stabilized Earth LRFD/QMP, Item SPV.0165
- Wall CIP Facing Mechanically Stabilized Earth LRFD, Item SPV.0165.
- Wall Wire Faced Mechanically Stabilized Earth LRFD, Item SPV.0165.
- Wall Wire Faced Mechanically Stabilized Earth LRFD/QMP, Item SPV.0165.
- *Wall Gabion LRFD, SPV under development.*

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17.1 Design Method

17.1.1 Design Requirements

All new structures and deck replacements are to be designed using *AASHTO LRFD Bridge Design Specifications*, hereafter referred to as *AASHTO LRFD*. Bridge rehabilitations and widenings are to be designed using either LFD or LRFD, at the designer's option.

LRFD utilizes load combinations called limit states which represent the various loading conditions which structural materials must be able to withstand. Limit states have been established in four major categories – strength, service, fatigue and extreme event. Different load combinations are used to analyze a structure for certain responses such as deflections, permanent deformations, ultimate strength and inelastic responses without failure. When all applicable limit states and combinations are satisfied, a structure is deemed acceptable under the LRFD design philosophy.

Bridges being designed with staged construction, whether new or rehabilitation, shall satisfy the requirements of LRFD (or LFD, if applicable) for each construction stage. Utilize the same load factors, resistance factors, load combinations, etc. as required for the final configuration, unless approved by Chief Structures Development Engineer at WisDOT.

17.1.2 Rating Requirements

Rating factors, RF, for inventory and operating rating are shown on the plans. Ratings will be based on *The Manual for Bridge Evaluation*, hereafter referred to as *AASHTO MBE*. See Chapter 45 – Bridge Rating for rating requirements. Existing ratings for rehabilitation projects where the final ratings will not change should be taken from HSI and placed on the final plans. See Section 6.2.2.3.4 for more information.

17.1.2.1 Standard Permit Design Check

New structures are also to be checked for the Wisconsin Standard Permit Vehicle (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. This truck will be designated as a Single Trip Permit Vehicle. It will have no escorts restricting the presence of other traffic on the bridge, 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.

See Chapter 45 – Bridge Rating for details about the Wisconsin Standard Permit Vehicle and calculating the maximum load for this permit vehicle.

17.2 LRFD Requirements

17.2.1 General

For superstructure member design, the component dimensions and the size and spacing of reinforcement shall be selected to satisfy the following equation for all appropriate limit states, as presented in **LRFD [1.3.2.1]**:

$$Q = \sum \eta_i \gamma_i Q_i \leq \phi R_n = R_r$$

Where:

η_i	=	Load modifier (a function of η_D, η_R, and η_I)
γ_i	=	Load factor
Q_i	=	Force effect: moment, shear, stress range or deformation caused by applied loads
Q	=	Total factored force effect
ϕ	=	Resistance factor
R_n	=	Nominal resistance: resistance of a component to force effects
R_r	=	Factored resistance = ϕR_n

17.2.2 WisDOT Policy Items

WisDOT policy items:

Set the value of the load modifier, η_i (see **LRFD [1.3.2.1]**), and its factors, η_D, η_R and η_I, all equal to 1.00.

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

17.2.3 Limit States

The following limit states (as defined in **LRFD [3.4.1]**) are utilized by WisDOT in the design of bridge superstructures.

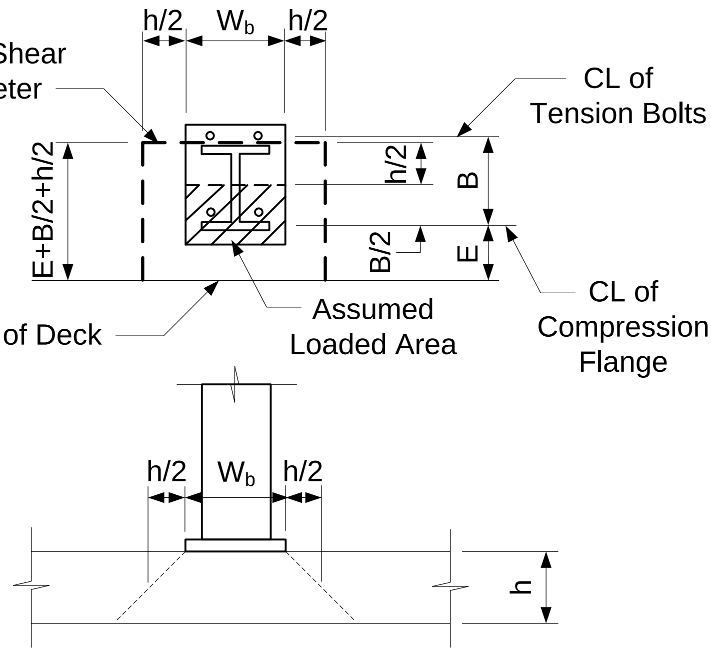


Figure 17.6-6
Assumed Load Distribution for Punching Shear

As used in Figure 17.6-6:

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

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

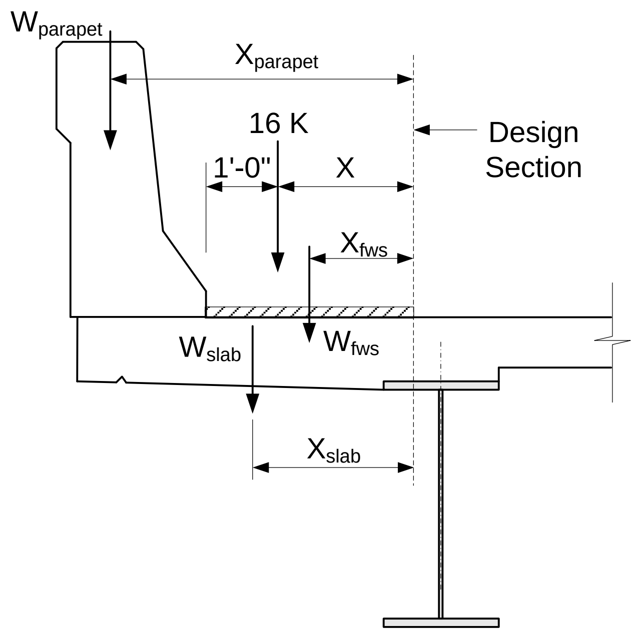


Figure 17.6-7
Design Case 3

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

Equivalent strip = 45.0 + 10.0X

Where:

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

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

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

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

17.6.1 Rail Loading for Slab Structures

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

17.6.2 WisDOT Overhang Design Practices

WisDOT policy item:

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

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

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

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

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

Table 17.6-2

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

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

Table 17.6-3

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

Effective Overhang (Feet)	Deck Thickness Between Girders, “t” (Inches)						
	8	8.5	9	9.5	10	10.5	11
1.75	0.277	0.277	0.277	0.277	0.251	0.202	0.159
2.00	0.287	0.287	0.287	0.287	0.264	0.220	0.180
2.25	0.295	0.295	0.295	0.295	0.274	0.234	0.198
2.50	0.302	0.302	0.302	0.302	0.282	0.246	0.212
2.75	0.307	0.307	0.307	0.307	0.290	0.255	0.224
3.00	0.312	0.312	0.312	0.312	0.295	0.278	0.263
3.25	0.394	0.394	0.394	0.394	0.392	0.389	0.340
3.50	0.465	0.465	0.465	0.465	0.464	0.436	0.412
3.75	0.497	0.497	0.497	0.497	0.477	0.489	0.480
4.00	0.567	0.567	0.567	0.567	0.542	0.501	0.504

Table 17.6-4
Reinforcing Steel (in^2/ft) for Cantilever Deck Slabs with Tubular Railing Type “NY”/”M”
Girder Type 1

Effective Overhang (Feet)	Deck Thickness Between Girders, “t” (Inches)						
	8	8.5	9	9.5	10	10.5	11
1.25	0.542	0.435	0.345	0.272	0.213	0.161	0.117
1.5	0.542	0.435	0.345	0.272	0.213	0.161	0.117
1.75	0.525	0.435	0.345	0.272	0.213	0.161	0.117
2	0.423	0.423	0.345	0.269	0.203	0.147	0.096
2.25	0.290	0.280	0.228	0.185	0.146	0.114	0.128
2.5	0.237	0.237	0.217	0.176	0.151	0.146	0.160
2.75	0.275	0.275	0.275	0.263	0.247	0.234	0.222
3	0.269	0.269	0.269	0.269	0.269	0.256	0.244
3.25	0.334	0.334	0.334	0.334	0.334	0.330	0.314

Table 17.6-5
Reinforcing Steel (in^2/ft) for Cantilever Deck Slabs with Tubular Railing Type “NY”/“M”
Girder Type 2

Notes:

1. Tables show the total area of transverse deck reinforcement required per foot.

2. The values in Table 17.6-2, Table 17.6-3, Table 17.6-4 and Table 17.6-5 are based on the following design criteria:

    - f'c = 4 ksi
    - fy = 60 ksi
    - Top steel clearance = 2 1/2"
    - Effective Overhang as illustrated in Figure 17.6-1

3. For Tubular Railing Type "NY"/"M", the No. 6 "U" bars located at the rail post locations should not be included when calculating the total available area of reinforcement.

4. The values in the shaded region are satisfied by the standard transverse reinforcement for all girder spacings and standard transverse deck reinforcement. No additional checks or reinforcement are required.

5. Details for additional overhang reinforcement are shown in Figure 17.6-8. Detail "A" shall be used with Table 17.6-2, Table 17.6-3 and Table 17.6-5. Detail "B" shall be used with Table 17.6-4.

6. For bridge decks with raised sidewalks, the additional reinforcement shown in Table 17.6-2, Table 17.6-3, Table 17.6-4, and Table 17.6-5, need not be used. See Standard Detail 17.01 – Median and Raised Sidewalk Details – for information pertaining to the additional reinforcement to be used at raised sidewalks.

Example Use of Tables:

Given Information:

54W" PSG, 15" from CL girder to Design Section -- (Girder Type 2)

Girder Spacing = 7'-0"

Overhang = 3'-0", Effective Overhang = 1'-9"

Type "NY" rail

From Table 17.5-1:

Deck thickness = 8"

Design Section at 15", use #5's @ 8.5", As provided = 0.43 in^2/ft

From Table 17.6-5:

Transverse area of steel required = 0.542 in^2/ft

Therefore:

Additional area of steel required = 0.542 - 0.43 = 0.112 in^2/ft

Use either one or two times the spacing of the standard transverse reinforcement.

Lapping every other bar: use #4's @ 17", As = 0.14 in^2/ft, use Detail "A".

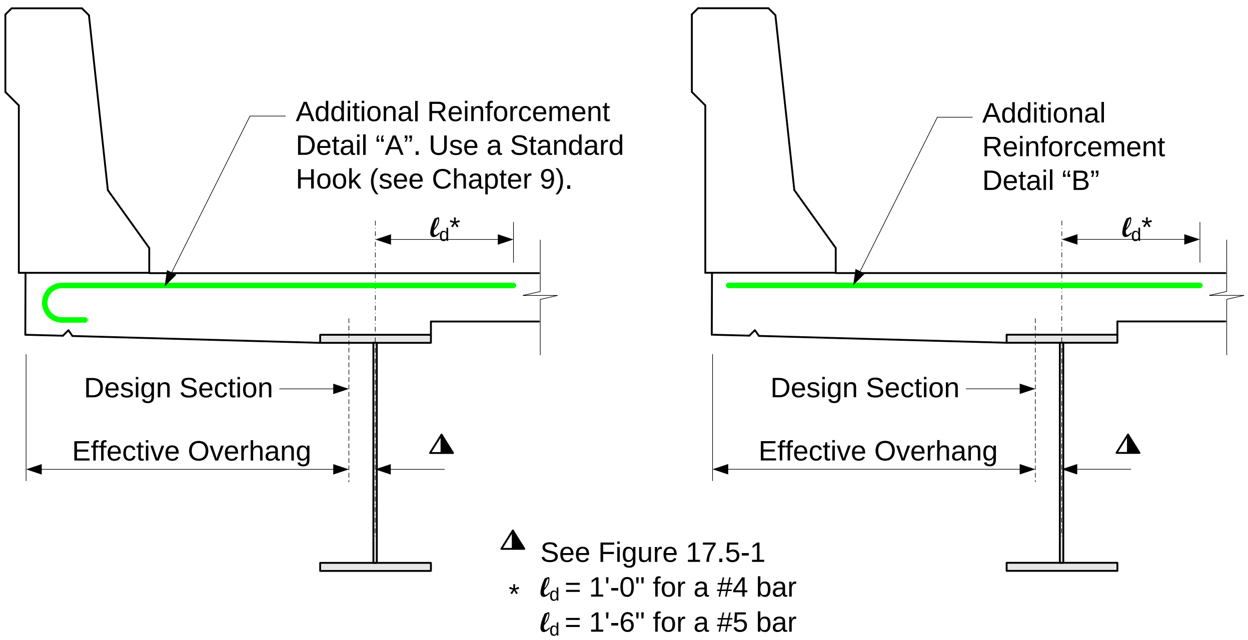


Figure 17.6-8
Overhang Reinforcement Details

To reiterate:

1. Details for additional overhang reinforcement are shown in Figure 17.6-8. Detail "A" shall be used with Table 17.6-2, Table 17.6-3 and Table 17.6-5. Detail "B" shall be used with Table 17.6-4.

2. Girder Type 1 shall include steel girders and the following prestressed girders; 28-inch, 36-inch, 36W-inch, 45W-inch. Prestressed girders used for rehabilitation projects, 45-inch, 54-inch and 70-inch, shall also be considered to be Girder Type 1. Girder Type 2 shall include the following prestressed girders; 54W-inch, 72W-inch and 82W-inch.

17.7 Construction Joints

Optional transverse construction joints are permitted on continuous concrete deck structures to limit the concrete volume in a single pour. Refer to the Standard Detail for Slab Pouring Sequence for the optimum slab pouring sequence. On steel structures over 300 feet long, transverse construction joints, if used, are to be placed at 0.6 of the span length beyond the pier in the direction of the pour. For continuous prestressed concrete girder bridges, optional transverse construction joints should be located midway between the cut-off points for continuity reinforcing steel or at 0.75 of the span, whichever is closest to the pier.

The rate of placing concrete for continuous steel girders shall equal or exceed 0.5 of the span length per hour but need not exceed 100 cubic yards per hour. Transverse construction joints may be omitted with approval of Bureau of Structures.

When the deck width of a girder superstructure exceeds 90 feet or the width of a slab superstructure exceeds 52 feet, a longitudinal construction joint with reinforcement through the joint shall be detailed. Longitudinal joints should be at least 6 inches from the edge of the top flange of the girder and preferably located beneath the median or parapet. Otherwise, the joint should be located along the edge of the lane line or in the middle of the lane. The longitudinal construction joint should be used for staged construction and for other cold joint applications within the deck. A longitudinal construction joint detail is provided in Standard Detail 17.02 – Deck and Slab Details.

Optional longitudinal construction joints shall be detailed accordingly in the plans. Longitudinal construction joints requested by the contractor are to be approved by the engineer. Optional and contractor requested joints are to be located as previously mentioned.

Open joints may be used in a median or between parapets. Considerations should be given to sealing open joints with compression seals or other sealants.

The structure plans should permit the contractor to propose an alternate construction joint schedule subject to approval of the engineer.

17.8 Bridge Deck Protective Systems

17.8.1 General

FHWA encourages states that require the use of de-icers to employ bridge deck protective systems. The major problem resulting in bridge deck deterioration is delamination of the concrete near the top mat of the reinforcing steel followed by subsequent spalling of the surface concrete. Research shows that the most prevalent cause of extensive deck deterioration is corrosion of the reinforcing steel due to the intrusion of chlorides into the concrete from repeated de-icer applications during snow and/or ice removal.

Several types of bridge deck protective systems are currently available. Some have been approved by FHWA based on their initial performance. Some of the more common types of protective systems are epoxy coated reinforcing steel, galvanized or stainless steel reinforcing steel, microsilica modified concrete or polymer impregnated concrete, cathodic protection and deck sealers. Epoxy coated reinforcing steel and deck sealers preferred by WisDOT.

Structures other than box culverts that are designed to carry an earth fill are required to have waterproofing membrane systems on the deck to protect the slab. This includes bridges designed for future grade changes.

17.8.2 Design Guidance

All deck reinforcement bars shall be epoxy coated and the top reinforcing bars shall have a minimum of 2 ½ inches of cover.

All decks shall receive a protective surface treatment. Other locations for protective surface treatment should include: parapet, parapet wing, median, sidewalk and edge of deck/slab and 1'-0" underside of deck/slab when open railings are utilized.

Additional protective systems may be desired to minimize future rehabilitations. One or a combination of systems may be used on large projects such as Mega Projects. Contact the WisDOT Bureau of Structures Design Section for approval and project specific guidance. The following systems are currently being used and should be considered on new structures and deck rehabilitations:

- High Performance Concrete (HPC) – This is typically used within the bridge superstructure (deck, diaphragms, parapets, structural approach slabs, etc.) on urban interchange projects

- Polymer overlays - This system extends the decks service life before rehabilitation is required.

- Stainless steel deck reinforcement – Use of stainless steel in lieu of epoxy bars may be justified for urban interchange projects and complex structures. Savings from reducing the number of rehabilitation projects and user costs can be substantial. Currently, only the enhanced corrosion protection benefits shall be utilized and reinforcement shall be selected per the epoxy coated deck design tables. The use of stainless reinforcing steel shall be approved by Chief Structures Development or Design Engineer and may require a life cycle analysis.

17.9 Bridge Approaches

The structure approach slab, or approach pavement, is part of the roadway design plans. Structure approach standards are provided in the *Facilities Development Manual* (*FDM*).

Guidance for the selection of pavement types for bridge approaches is as shown in *FDM* Procedure 14-10-15.

Considerations for site materials, drainage and backfill are provided in Chapter 12 – Abutments. Most approach pavement failures are related to settlement of embankment or foundation materials. Past experience shows that significant settlement is most likely to occur where marginal materials are used. Designers are encouraged to provide perforated underdrains wrapped in geotextile fabric placed in a trench filled with crushed stone. Also, abutment backfill material should be granular in nature and consolidated under optimum moisture conditions.

17.10 Design of Precast Prestressed Concrete Deck Panels

17.10.1 General

An advantage of stay-in-place forms is that they can be placed in less time than it takes to place the forms for a conventional deck. There is also a labor savings because the extra step of removing deck forms is not required. Stay-in-place forms are often the preferred system for shallow box girders because of the difficulty of removing forms in a confined space.

If not detailed in the contract documents, precast concrete deck panels may be used at the option of the contractor, provided the specifications permit their use. A standardized special provision (STSP) for optional use of precast prestressed concrete deck plans is available from the Bureau of Highway Construction, Standards Development Section.

When a contractor elects to use precast deck panels at their option, the contractor is responsible for the shop plans of the panels and any other changes that may be required to the reinforcing steel in the cast-in-place portion of the deck. Payment to a contractor who chooses to use stay-in-place forms is based on the contract prices bid for the conventional cast-in-place deck.

Deck panels are only used between the inside faces of the exterior girders. The overhangs outside the exterior girders are formed and the concrete placed in the same way as in a conventional cast-in-place deck. On skewed decks, the contractor may form and cast the skewed portion of the deck full depth or they may use skewed end deck panels which may be individually precast or saw-cut from square end planks.

A problem with decks formed with concrete deck panels is that cracks often form in the cast-in-place concrete over the transverse joints between panels and along the edges of the panels parallel to the girders. Reflection cracking is less of a problem when these panels are used on prestressed concrete girders than on steel girders. Simple-span prestressed concrete girder bridges have less reflective cracking than continuous-span prestressed concrete girder bridges.

17.10.2 Deck Panel Design

The design of precast prestressed concrete deck panels shown in Table 17.10-1 is based on *AASHTO LRFD* design criteria. These panels were designed for flexure due to the HL-93 design truck live load, dead load of the plastic concrete supported by the panels, a construction load of 50 psf, dead load of the panels and a future wearing surface of 20 psf. The live load moments were obtained from **LRFD [Table A4-1]**.

At the request of precast deck panel fabricators, only two strand sizes are used – 3/8 inch and 1/2 inch. Precast deck panel fabricators do not want the additional overhead expense of stocking 7/16-inch strand. Strand spacing is given in multiples of 2 inches.

WisDOT exception to AASHTO:

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

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

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

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

The allowable tensile stress in the panels, as presented in **LRFD [Table 5.9.4.2.2-1]**, is as follows:

$$0.0948\sqrt{f'_c}$$

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

The transfer length of the strands is assumed to be 60 strand diameters at a stress of 202.5 ksi. The development length, L_d, of the strands, as presented in **LRFD [5.11.4.2]**, is assumed to be as follows:

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

Where:

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

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

L_d = Development length beyond critical section (inches)

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

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

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

LRFD [5.7.3.3.2] requires that the moment capacity of a flexural member be greater than the cracking moment based on the modulus of rupture. This requirement may be waived if the moment capacity is greater than 1.33 times the factored design moment. The purpose of this requirement is to provide a minimum amount of reinforcement in a flexural member so that a flexural failure will not be sudden or occur without warning. Tests have shown that for slabs on girders, the failure mode is a punching shear failure and not a flexural failure. ACI 10.5.4 also recognizes the difference between slabs and beams and does not require the same minimum reinforcement for slabs. For these reasons, **LRFD [5.7.3.3.2]** was not considered in the designs of the panels shown in Table 17.10-1. However, panels with a width of 6 feet or more meet the requirements of **LRFD [5.7.3.3.2]**.

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

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

Crack control was checked in accordance with **LRFD [5.7.3.4]** and as shown in 17.5.3.1. A concrete strength of 4 ksi was assumed, and the haunch height over the girders was not considered.

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

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

17.10.3.1 Longitudinal Reinforcement

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

17.10.4 Details

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

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

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

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

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

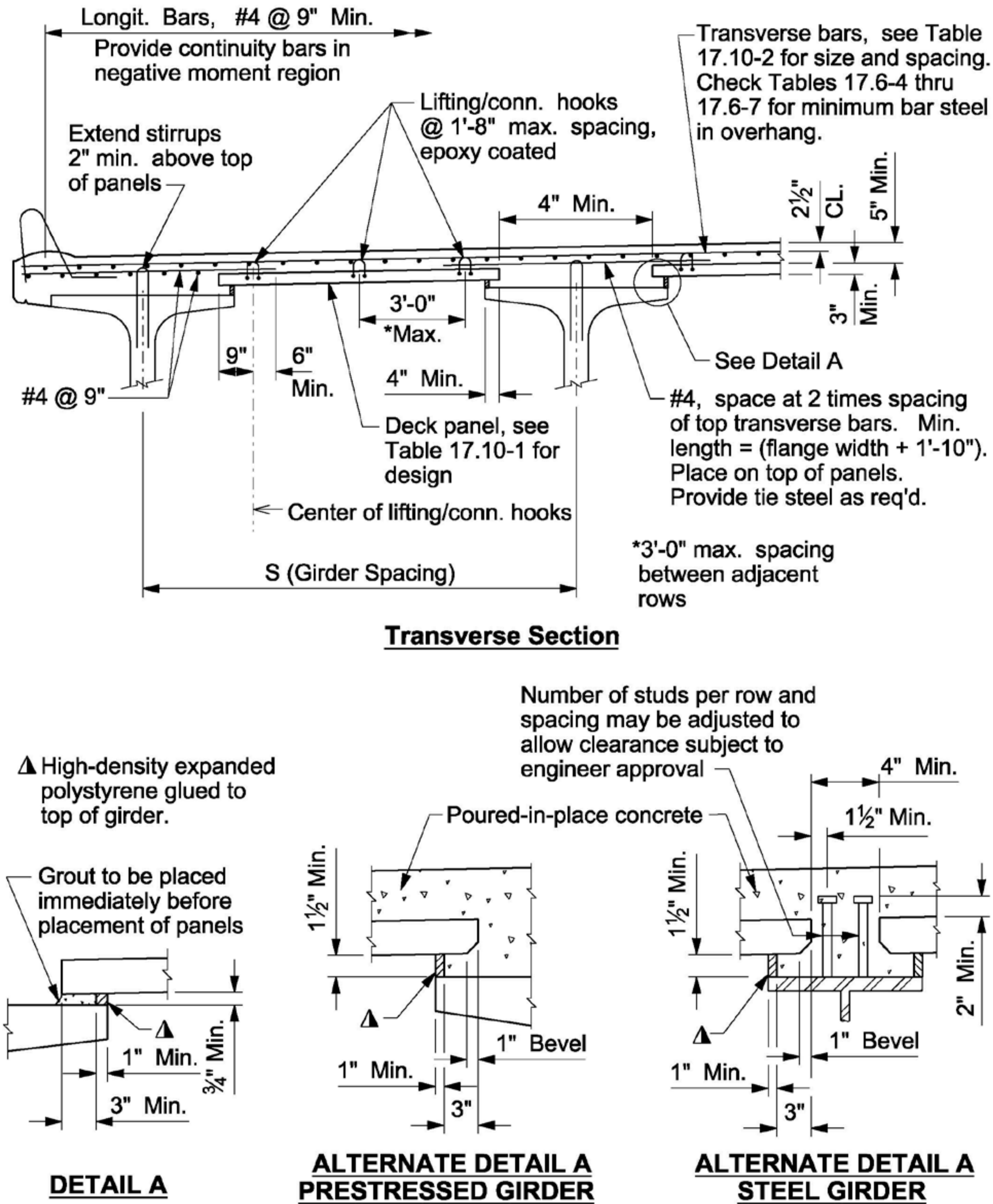
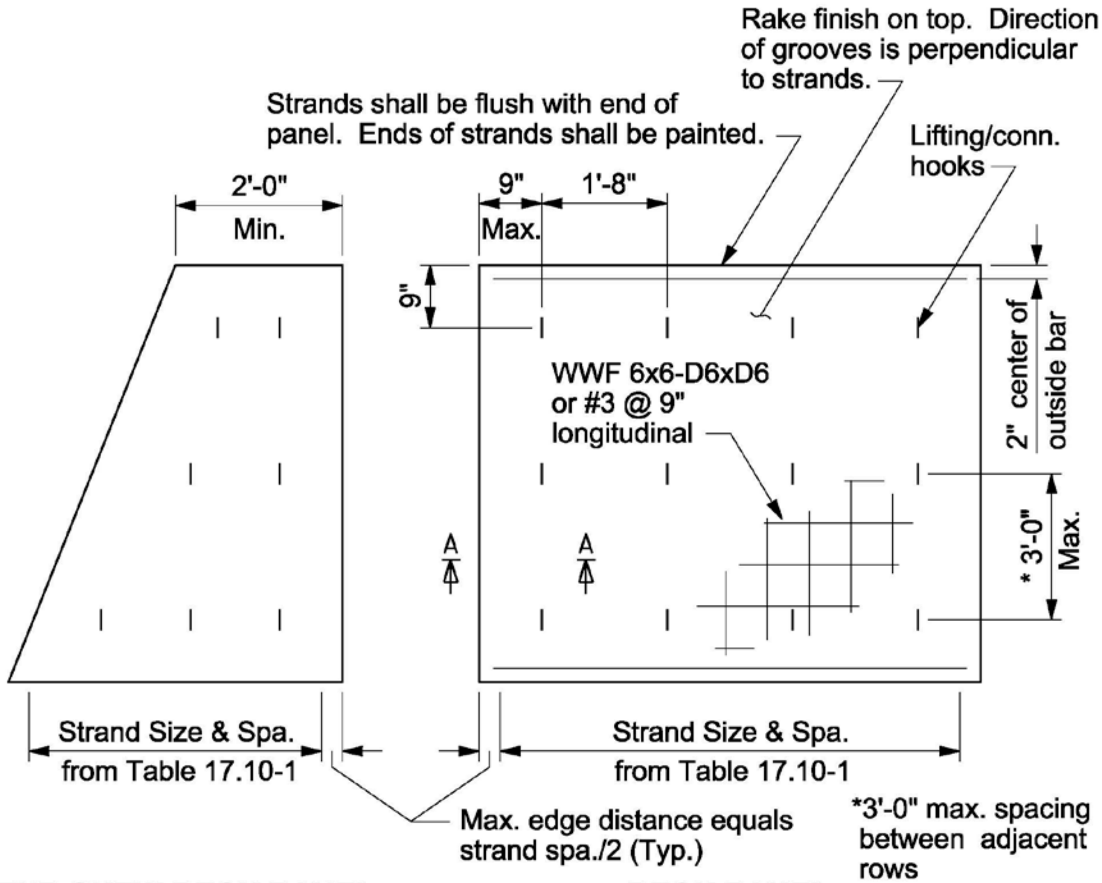
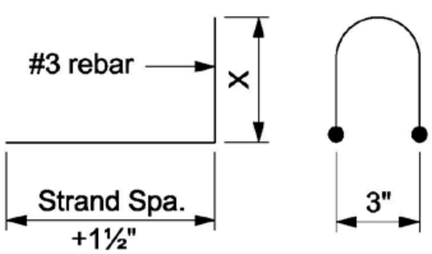


Figure 17.10-1
Transverse Section through Slab on Girders with Deck Panel and Details



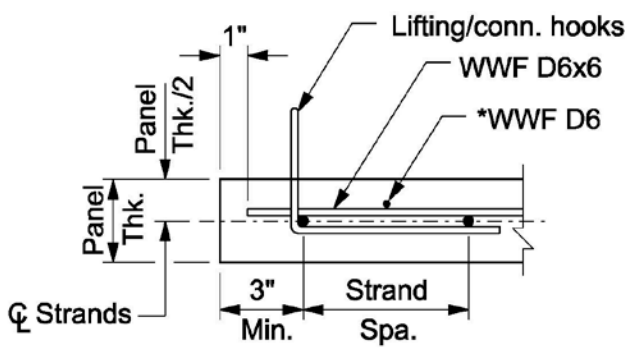
END SKEW DECK PANEL

DECK PANEL



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

Lifting/Conn. Hook Detail



Part Section A-A

*Bars in WWF which are parallel to the strands must be a minimum of 1" clear from the strands.

Figure 17.10-2
Deck Panel Details

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

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

Notes:

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

Table 17.10-1

Precast Prestressed Concrete Deck Panel Design Table

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

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

Notes:

- Designed per AASHTO LRFD with HL-93 Loading.
- f'c deck = 4.0 ksi
- fy = 60 ksi
- Steel is 2 ½" clear from top of slab. Designed for 20 psf future wearing surface. "Total Slab Thickness" includes thickness of deck panel and poured in place concrete.

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

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E18-1 Continuous 3-Span Haunched Slab - LRFD

A continuous 3-span haunched slab structure is used for the design example. The same basic procedure is applicable to continuous flat slabs. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. Design using a slab width equal to one foot. (***Example is current through LRFD Seventh Edition***)

E18-1.1 Structure Preliminary Data

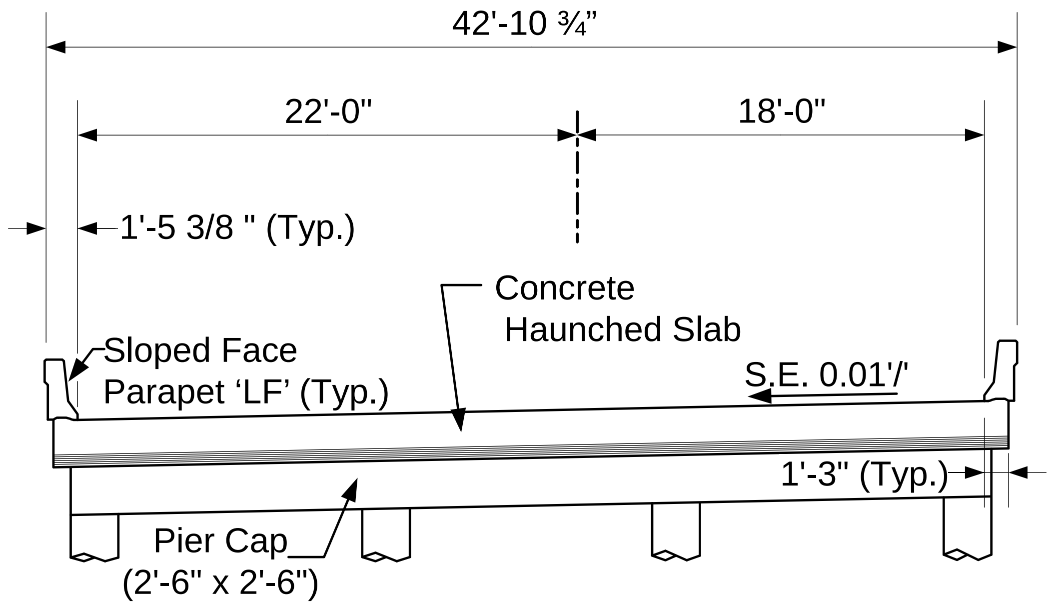

Figure E18.1

Section Perpendicular to Centerline

Live Load: HL-93
(A1) Fixed Abutments at both ends
Parapets placed after falsework is released

Geometry:

$L_1 := 38.0$ ft — Span 1

$L_2 := 51.0$ ft — Span 2

$L_3 := 38.0$ ft — Span 3

$slab_{width} := 42.5$ ft — out to out width of slab

$skew := 6$ deg — skew angle (RHF)

$w_{roadway} := 40.0$ ft — clear roadway width

Material Properties: (See 18.2.2)

$f'_c := 4$ ksi — concrete compressive strength

$f_y := 60$ ksi yield strength of reinforcement

$E_c := 3800$ ksi modulus of elasticity of concrete

$E_s := 29000$ ksi modulus of elasticity of reinforcement

$n := 8$ E_s / E_c (modular ratio)

Weights:

$w_c := 150$ pcf concrete unit weight

$w_{LF} := 387$ plf weight of Type LF parapet (each)

E18-1.2 LRFD Requirements

For concrete slab design, the slab dimensions and the size and spacing of reinforcement shall be selected to satisfy the equation below for all appropriate Limit States: (See 18.3.2.1)

$$Q = \Sigma\eta_i \cdot \gamma_i \cdot Q_i \le \phi \cdot R_n = R_r \qquad \text{(Limit States Equation)}$$

The value of the load modifier is:

$\eta_i := 1.0$ for all Limit States (See 18.3.2.2)

The force effect, Q_i , is the moment, shear, stress range or deformation caused by applied loads.

The applied loads from **LRFD [3.3.2]** are:

DC = dead load of slab (DC_{slab}), ½ inch wearing surface ($DC_{1/2"WS}$) and parapet dead load (DC_{para}) - (See E18-1.3)

DW = dead load of future wearing surface (DW_{FWS}) - (See E18-1.3)

LL+IM = vehicular live load (LL) with dynamic load allowance (IM) - (See E18-1.4)

The Influence of ADTT and skew on force effects, Q_i , are ignored for slab bridges (See 18.3.2.2).

The values for the load factors, γ_i , (for each applied load) and the resistance factors, ϕ , are found in Table E18.1.

The total factored force effect, Q , must not exceed the factored resistance, R_r . The nominal resistance, R_n , is the resistance of a component to the force effects.

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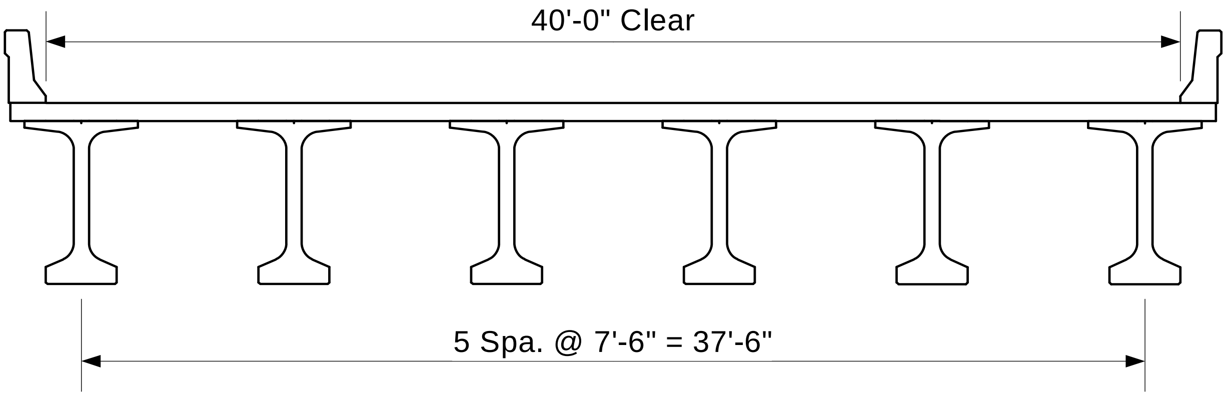
E19-1 Single Span Bridge, 72W" Prestressed Girders - LRFD

This example shows design calculations for a single span prestressed gider bridge. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. ***(Example is current through LRFD Seventh Edition)***

E19-1.1 Design Criteria



$L := 146$	center to center of bearing, ft
$L_g := 147$	total length of the girder (the girder extends 6 inches past the center of bearing at each abutment).
$w_b := 42.5$	out to out width of deck, ft
$w := 40$	clear width of deck, 2 lane road, 3 design lanes, ft
$f'_c := 8$	girder concrete strength, ksi
$f'_{ci} := 6.8$	girder initial concrete strength, ksi New limit for release strength.
$f'_{cd} := 4$	deck concrete strength, ksi
$f_{pu} := 270$	low relaxation strand, ksi
$d_b := 0.6$	strand diameter, inches
$A_s := 0.217$	area of strand, in^2
$w_p := 0.387$	weight of Wisconsin Type LF parapet, klf
$t_s := 8$	slab thickness, in
$t_{se} := 7.5$	effective slab thickness, in
$skew := 20$	skew angle, degrees
$E_s := 28500$	ksi, Modulus of Elasticity of the Prestressing Strands
$w_c := 0.150$	kcf

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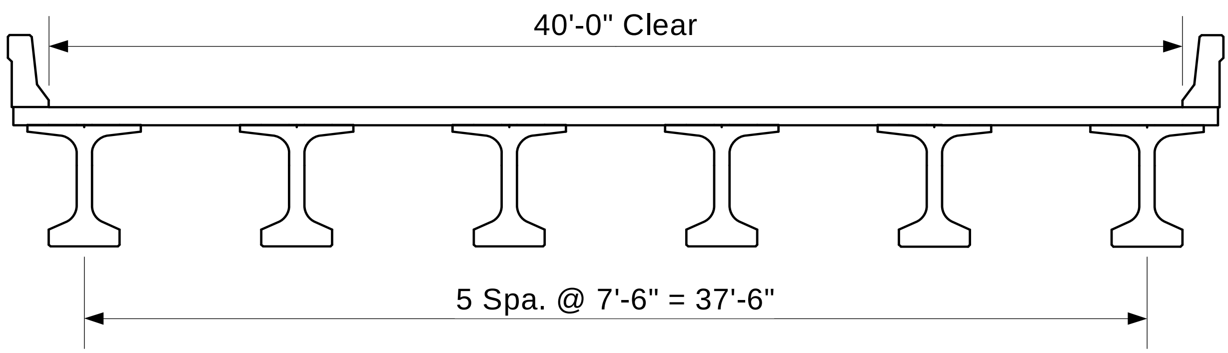
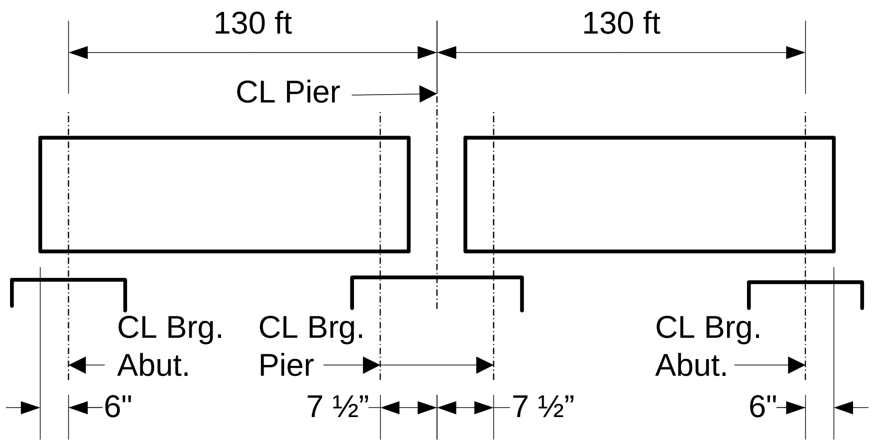
E19-2 Two-Span 54W" Girder, Continuity Reinforcement - LRFD

This example shows design calculations for the continuity reinforcement for a two span prestressed girder bridge. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. ***(Example is current through LRFD Seventh Edition)***

E19-2.1 Design Criteria





$L := 130$	center of bearing at abutment to CL pier for each span, ft
$L_g := 130.375$	total length of the girder (the girder extends 6 inches past the center of bearing at the abutment and 1.5" short of the center line of the pier).
$w_b := 42.5$	out to out width of deck, ft
$w := 40$	clear width of deck, 2 lane road, 3 design lanes, ft
$f'_c := 8$	girder concrete strength, ksi
$f'_{cd} := 4$	deck concrete strength, ksi
$f_y := 60$	yield strength of mild reinforcement, ksi

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E19-3 Box Section Beam

This example shows design calculations for a single span prestressed box multi-beam bridge having a 2" concrete overlay and is designed for a 20 pound per square foot future wearing surface. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. ***(Example is current through LRFD Seventh Edition)***

E19-3.1 Preliminary Structure Data

Design Data
A-1 Abutments at both ends
Skew: 0 degrees
Live Load: HL-93
Roadway Width: 28 ft. minimum clear

$L := 44$	Span Length, single span, ft
$L_g := 44.5$	Girder Length, the girder extends 3" past the CL bearing at each abutment, single span, ft
$N_L := 2$	Number of design lanes
$t_{overlay} := 2$	Minimum overlay thickness, inches
$f_{pu} := 270$	Ultimate tensile strength for low relaxation strands, ksi
$d_s := 0.5$	Strand diameter, inches
$A_s := 0.1531$	Area of prestressing strands, in^2
$E_s := 28500$	Modulus of elasticity of the prestressing strands, ksi
$f'_c := 5$	Concrete strength (prestressed box girder), ksi
$f'_{ci} := 4.25$	Concrete strength at release, ksi
$K_1 := 1.0$	Aggregate correction factor
$w_c := 0.150$	Unit weight of concrete for box girder, overlay, and grout, kcf
$fy := 60$	Bar steel reinforcement, Grade 60, ksi.
$w_{rail} := 0.075$	Weight of Type "M" rail, klf
$Wh_{rail} := 0.42$	Width of horizontal members of Type "M" rail, feet
$\mu := 0.20$	Poisson's ratio for concrete, **LRFD [5.4.2.5]**

Based on past experience, the modulus of elasticity for the precast concrete are given in Chapter 19 as $E_{beam6} := 5500$ ksi for a concrete strength of 6 ksi. The values of E for different concrete strengths are calculated as follows:

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Length (ft)	0.0	19.0	38.0	57.0	76.0	95.0
Steel Weight	0	400	663	789	778	630
Additional Miscellaneous Steel	0	166	278	336	340	290
Cast 1	0	1190	1994	2413	2447	2096
Cast 2	0	-29	-58	-87	-116	-145
Cast 3	0	25	51	76	102	127
Sum of Casts	0	1186	1987	2402	2433	2078
Deck & Haunches (Simultaneous Cast)	0	1184	1983	2396	2424	2067

Table 24.12-1

Moments from Deck Placement Analysis (K-ft)

The slight differences in the moments on the last line of Table 24.12-1 (assuming a simultaneous placement of the entire slab) and the sum of the moments due to the three casts are due to the changes in the girder stiffness with each sequential cast. The principle of superposition does not apply directly in the deck-placement analyses, since the girder stiffness changes at each step of the analysis. Although the differences in the moments are small in this example, they can be significantly greater depending on the span configuration. The effects of the deck placement sequence must be considered during design.

In regions of positive flexure, the non-composite girder should be checked for the effect of the maximum accumulated deck-placement moment. This moment at 76 feet from Abutment 1 is computed as:

$$M = 778 + 340 + 2{,}447 = 3{,}565 \text{ kip-ft}$$

This value agrees with the moment at this location shown in Figure 24.12-4.

In addition to the dead load moments during the deck placement, unfactored dead load deflections and reactions can also be investigated similarly during the construction condition.

When investigating reactions during the construction condition, if uplift is found to be present during deck placement, the following options can be considered:

- Rearrange the concrete casts.
- Specify a temporary load over that support.
- Specify a tie-down bearing.
- Perform another staging analysis with zero bearing stiffness at the support experiencing lift-off.

24.13 Painting

The final coat of paint on all steel bridges shall be an approved color. Exceptions to this policy may be considered on an individual basis for situations such as scenic river crossings, unique or unusual settings or local community preference. The Region is to submit requests for an exception along with the Structure Survey Report. The colors available for use on steel structures are shown in Chapter 9 - Materials.

Unpainted weathering steel is used on bridges over streams and railroads. All highway grade separation structures require the use of painted steel, since unpainted steel is subject to excessive weathering from salt spray distributed by traffic. On weathering steel bridges, the end 6' of any steel adjacent to either side of an expansion joint and/or hinge is required to have two shop coats of paint. The second coat is to be brown color similar to rusted steel. Do not paint the exterior face of the exterior girders for aesthetic reasons, but paint the hanger bar on the side next to the web. Additional information on painting is presented in Chapter 9 - Materials.

For painted steel plate I-girders utilize a three-coat system defined by the Standard Specification bid item “Painting Epoxy System (Structure)”. For painted tub girders use a two-coat system defined by the STSP “Painting Polysiloxane System (Structure)”, which includes painting of the inside of the tubs.

Paint on bridges affects the slip resistance of bolted connections. Since faying surfaces that are not galvanized are typically blast-cleaned as a minimum, a Class A surface condition should only be used to compute the slip resistance when Class A coatings are applied or when unpainted mill scale is left on the faying surface. Most commercially available primers will qualify as Class B coatings.

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E27-1 DESIGN EXAMPLE - STEEL REINFORCED ELASTOMERIC BEARING

ThIs design example is for a 3-span prestressed girder structure. The piers are fixed supports and the abutments accommodate expansion.
(Example is current through LRFD Seventh Edition)

E27-1.1 Design Data

Bearing location: Abutment (Type A3)

Girder type: 72W

$L_{exp} := 220$ Expansion length, ft

$b_f := 2.5$ Bottom flange width, ft

$DL_{serv} := 167$ Service I limit state dead load, kips

$DL_{ws} := 23$ Service I limit state future wearing surface dead load, kips

$LL_{serv} := 62$ Service I limit state live load, kips

$h_{rcover} := 0.25$ Elastomer cover thickness, in

$h_s := 0.125$ Steel reinforcement thickness, in

$F_y := 36$ Minimum yield strength of the steel reinforcement, ksi

Temperature Zone:	C (Southern Wisconsin)	**LRFD [Fig. 14.7.5.2-1]**
Minimum Grade of Elastomer:	3	**LRFD [Table 14.7.5.2-1]**
Elastic Hardness:	Durometer 60 +/- 5	(used 55 for design)
Shear Modulus (G):	0.1125 ksi < G <0.165 ksi	**LRFD [Table 14.7.6.2-1]**
Creep Deflection @ 25 Years		
divided by instantaneous deflection:	0.3	**LRFD [Table 14.7.6.2-1]**

E27-1.2 Design Method

Use Design Method A **LRFD [14.7.6]**
Method A results in a bearing with a lower capacity than a bearing designed using Method B. However the increased capacity resulting from the use of Method B requires additional testing and quality control.

E27-1.3 Dynamic Load Allowance

The influence of impact need not be included for bearings **LRFD [14.4.1]**; however, dynamic load allowance will be included to follow a **WisDOT policy item**.

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30.1 Crash-Tested Bridge Railings and FHWA Policy

Crash test procedures for full-scale testing of guardrails were first published in 1962 in the *Highway Research Correlation Services Circular 482*. This was a one-page document that specified vehicle mass, impact speed, and approach angle for crash testing and was aimed at creating uniformity to traffic barrier research between several national research agencies.

In 1974, the National Cooperative Highway Research Program (NCHRP) published their final report based on NCHRP Project 22-2, which was initiated to address outstanding questions that were not covered in *Circular 482*. The final report, NCHRP Report 153 – *"Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances,"* was widely accepted following publication; however, it was recognized that periodic updating would be required.

NCHRP Project 22-2(4) was initiated in 1979 to address major changes to reflect current technologies of that time and NCHRP Report 230, *"Recommended Procedures for the Safety Performance Evaluation of Highway Safety Appurtenances,"* was published in 1980. This document became the primary reference for full-scale crash testing of highway safety appurtenances in the U.S. through 1993.

In 1986, the Federal Highway Administration (FHWA) issued a policy memorandum that stated highway bridges on the National Highway System (NHS) and the Interstate Highway System (IHS) must use crash-tested railings in order to receive federal funding.

The American Association of State Highway and Transportation Officials (AASHTO) recognized that the evolution of roadside safety concepts, technology, and practices necessitated an update to NCHRP Report 230 approximately 7 years after its adoption. NCHRP Report 350, *"Recommended Procedures for the Safety Performance Evaluation of Highway Features,"* represented a major update to the previously adopted report. The updates were based on significant changes in the vehicle fleet, the emergence of many new barrier designs, increased interest in matching safety performance to levels of roadway utilization, new policies requiring the use of safety belts, and advances in computer simulation and other evaluation methods.

NCHRP Report 350 differs from NCHRP Report 230 in the following ways: it is presented in all-metric documentation, it provides a wider range of test procedures to permit safety performance evaluations for a wider range of barriers, it uses a pickup truck as the standard test vehicle in place of a passenger car, it defines other supplemental test vehicles, it includes a broader range of tests to provide a uniform basis for establishing warrants for the application of roadside safety hardware that consider the levels of use of the roadway facility, it includes guidelines for selection of the critical impact point for crash tests on redirecting-type safety hardware, it provides information related to enhanced measurement techniques related to occupant risk, and it reflects a critical review of methods and technologies for safety-performance evaluation.

In May of 1997, a memorandum from Dwight A. Horne, the FHWA Chief of the Federal-Aid and Design Division, on the subject of "Crash Testing of Bridge Railings" was published. This memorandum identified 68 crash-tested bridge rails, consolidated earlier listings, and 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, with the exception of the type “F” steel railing, have been approved by FHWA per the crash tests as recommended in NCHRP Report 350. In order to use railings other than Bridge Office Standards, the railings must conform to MASH or must be crash tested rails which are available from the FHWA office. Any railings that are not crash tested must be reviewed by FHWA when they are used on a bridge, culvert, retaining wall, etc.

WisDOT 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 or NCHRP Report 350 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, 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 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.

A “SS” or solid parapet shall be used on all grade separation structures and railroad crossings to minimize snow removal falling on the traffic below.

2. The sloped face parapet "LF" and “HF” parapets shall be used as Traffic Railings for rehabilitation projects only to match the existing parapet type. The sloped face parapets were crash-tested per NCHRP Report 230 and meet NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

3. The “51F” parapet shall only be used as a Traffic Railing on the median side of a structure when it provides a continuation of an approach 51 inch high median barrier.

4. The vertical face parapet “A” can be used for all design speeds. The vertical face parapet is recommended for use as a Combination Railing on raised sidewalks or as a Traffic Railing where the design speed is 45 mph or less. If the structure has a raised sidewalk on one side only, a sloped parapet should be used on the side opposite of the sidewalk. For design speeds exceeding 45 mph, at locations where the parapet is protected by a Traffic Railing between the roadway and a sidewalk at grade, the vertical face parapet can be used as a Pedestrian Railing. Under some circumstances, the vertical face parapet “A” can be used as a Traffic Railing for design speeds exceeding 45 mph with the approval of the Bureau of Structures Development Section. The vertical face parapet “A” was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

5. Aesthetic railings may be used if crash tested according to Section 30.1 or follow the guidance provided in Section 30.4.

   The Texas style aesthetic parapet, type “TX”, can be used as a Combination Railing or Traffic Railing on raised sidewalks on structures with a design speed of 45 mph or less. For design speeds exceeding 45 mph, at locations where the parapet is protected by a Traffic Railing between the roadway and a sidewalk at grade, the type “TX” parapet can be used. This parapet is very expensive; however, form liners simulating the openings can be used to reduce the cost of this parapet. The type “TX” parapet was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-2 based on a May 1997 FHWA memorandum.

6. The type “PF” tubular railing, as shown in the Standard Details, shall not be used on bridge plans with a PS&E after 2013. This railing was not allowed on the National Highway System (NHS). The type “PF” railing was used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less.

7. Combination Railings, type “C1” through “C6”, are shown in the Standard Details and are approved as aesthetic railings attached to concrete parapets. The aesthetic additions are placed at least 5” from the crash-tested rail face per the Standard Details and have previously been determined to not present a snagging potential. These railings can only be used when the design speed is 45 mph or less, or the railing is protected by a Traffic Railing between the roadway and a sidewalk at grade. The crash test criteria of the combination railings are based on the concrete parapets

to which they are attached (i.e., if a type “C1” combination railing is attached to the top of a vertical face parapet type “A”, the railing meets crash test criteria for TL-4).

8. Chain Link Fence and 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 was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

10. Timber Railing as shown in the Standard Details is not allowed on the National Highway System (NHS). Timber Railing may be used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less. The Timber Railing was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-2 based on a May 1997 FHWA memorandum.

11. The type "W" railing, as shown in the Standard Details, is not allowed on the National Highway System (NHS). This railing may be used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less. The type “W” railing shall be used on concrete slab structures only. The use of this railing on girder type structures has been discontinued. Generally, type "W" railing is considered when the roadway approach requires standard beam guard and if the structure is 80 feet or less in length. The type “W” railing was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-3 based on a May 1997 FHWA memorandum.

12. Type “M” steel railing, as shown in the Standard Details, shall generally be used as a Traffic Railing on all functional classes of Wisconsin structures with a design speed of 45 mph or less. The type “M” railing may be used on roadways with a design speed exceeding 45 mph where the minimum 0.5% deck grade cannot be accommodated for proper drainage based on project specific constraints. The type “M” railing also can be used in place of the type “W” railing when placed on girder type structures as type “W” railings are not allowed for this application. However, the type “M” railing is

not allowed for use on prestressed box girder bridges. This railing shall be considered where the Region requests an open railing. The type “M” railing was crash-tested per NCHRP Report 350 and meets criteria for TL-4.

13. 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 were crash-tested per NCHRP Report 350 and meet criteria for TL-4.

14. The type "F" steel railing, as shown in the Standard Details, shall not be used on new bridge plans with a PS&E after 2013. It has not been allowed on the National Highway System (NHS) in the past and was used on non-NHS roadways with a design speed of 45 mph or less.

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 Section 30.2 (i.e., cast-in-place anchors are used at exterior parapet location). See Standard Details 30.10 and 30.14 for more information.

3. Sign structures, sign trusses, and monotubes shall be placed on top of railings to meet the working width and zone of intrusion dimensions noted in *FDM 11-45 Section 2.3.6.2.2* and *Section 2.3.6.2.3* respectively.

4. It is desirable to avoid attaching noise walls to bridge railings. However, in the event that noise walls are required to be located on bridge railings, compliance with the setback requirements stated in Section 30.4 and what is required in *FDM 11-45 Sections 2.3.6.2.2* and *2.3.6.2.3* is not required. *Note: WisDOT is currently investigating the future use of noise walls on bridge structures in Wisconsin.*

5. Temporary bridge barriers shall be designed in accordance with *FDM SDD 14b7*. Where temporary bridge barriers are being used for staged construction, the designer should attempt to meet the required offsets so that the barrier does not require anchorage which would necessitate drilling holes in the new deck.

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

7. Refer to Standard Detail 30.07 – Vertical Face Parapet “A” – for detailing concrete parapet or sidewalk deflection joints. These joints are used based on previous experience with transverse deck cracking beneath the parapet joints.

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

9. For beam guard type “W” railing, locate the expansion splice at a post or on either side of the expansion joint.

10. Sidewalks - If there is a Traffic Railing between the roadway and an at grade sidewalk, and the roadway side of the Traffic Railing is more than 11’-0” from the exterior edge of deck, access must be provided to the at grade sidewalk for the snooper truck to inspect the underside of the bridge. The sidewalk width must be

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 formliner patterns shown in Standard Detail 4.01 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.

<u>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</u>:

1. All Traffic Railings shall meet the crash testing guidelines outlined in Section 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 Standard Details 30.17 and 30.18 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 formliner patterns shown in Standard Detail 4.01 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.

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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)***

E36-1.1 Design Criteria

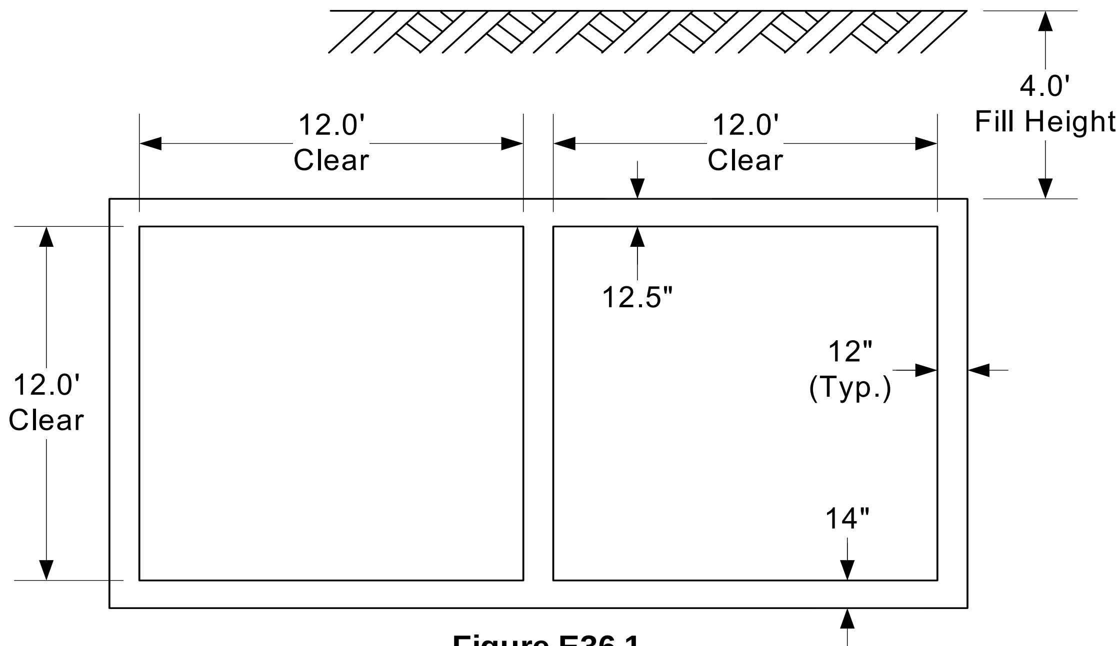

Figure E36.1
Box Culvert Dimensions

$NC = 2$	number of cells
$Ht = 12.0$	cell clear height, ft
$W_1 = 12.0$	cell 1 clear width, ft
$W_2 = 12.0$	cell 2 clear width, ft
$L = 134.0$	culvert length, ft
$t_{ts} = 12.5$	top slab thickness, in
$t_{bs} = 14.0$	bottom slab thickness, in
$t_{win} = 12.0$	interior wall thickness, in
$t_{wex} = 12.0$	exterior wall thickness, in
$H_{apron} := Ht + \frac{t_{ts}}{12}$	apron wall height above floor, ft

$$H_{apron} = 13.04$$

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39.1 General

WisDOT policy item:

The design for sign structures shall be in accordance with the *AASHTO Standard Specifications for Highway Bridges, 17th Edition*.

Signing is an integral part of the highway plan and as such is developed with the roadway and bridge design. Aesthetic as well as functional considerations are essential to sign structure design. Supporting sign structures should exhibit clean, light, simple lines which do not distract the motorist or obstruct his view of the highway. In special situations sign panels may be supported on existing or proposed grade separation structures in lieu of an overhead sign structure. Aesthetically this is not objectionable if the sign does not extend below the girders or above the top of the parapet railing. Some of the more common sign support structures are shown in the following figure.

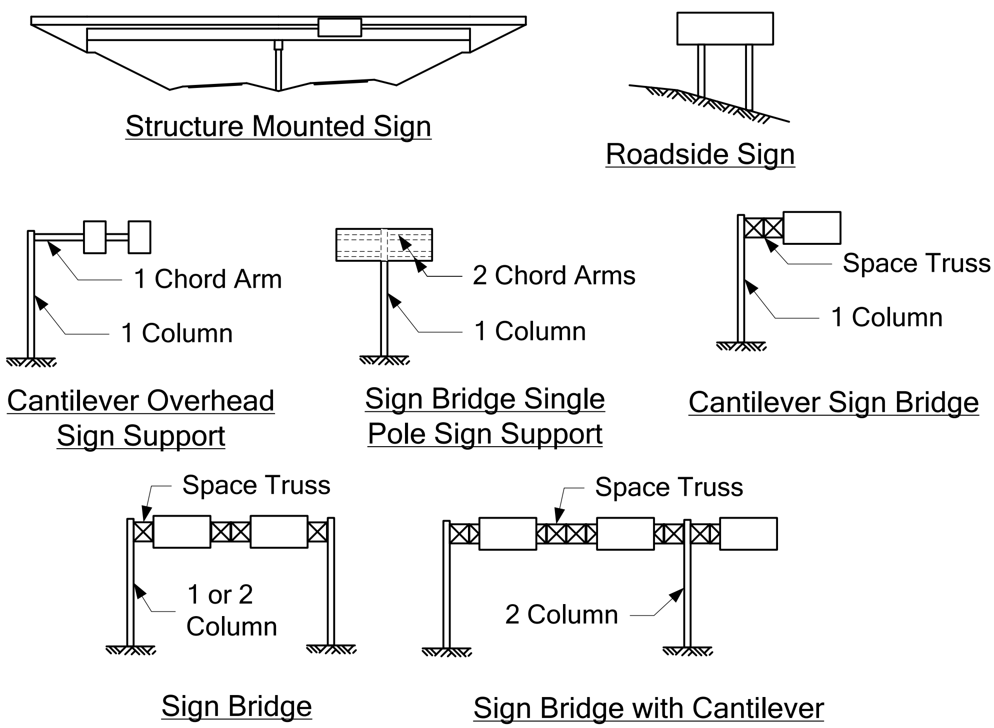

Figure 39.1-1
Sign Support Structures

39.1.1 Signs on Roadway

Roadside sign supports are located behind existing or planned guardrail as far as practical from the roadway out of the likely path of an errant vehicle. If roadside signs are located within the 30 foot corridor and not protected, break-away sign supports are detailed. Wisconsin has experienced that the upper hinge on ground mounted signs with break-away supports does not work and it is not used. Since FHWA has not approved this removal, the hinge is used on all federal projects. DMS, which includes both dynamic message signs and variable message signs, roadside sign type supports are to be protected by concrete barrier or guardrail. All overhead sign-column type supports are located at the edge of shoulder adjacent to the traveled roadway or placed behind barrier type guardrail. See the Facilities Development Manual (FDM) 11-55-20.5 for details on shielding requirements. When protection is impractical or not desirable, the uprights shall be designed with applicable extreme event collision loads in accordance to Section 13.4.10 of this manual.

Overhead sign structures, for new and replacement structures only, are to have a minimum vertical clearance of 18'-3" above the roadway. See FDM, Procedure 11-35-1 Attachment 1.9, for clearances relating to existing sign structures. The minimum vertical clearance is set slightly above the clearance line of the overpass structure for signs attached to existing structures.

39.1.2 Signs Mounted on Structures

Signs are typically installed along the major axes of a structure. Wisconsin has allowed sign attachment up to a maximum of a 20 degree skew. Any structure with greater skew requires mounting brackets to attach signs perpendicular to the roadway.

39.1.2.1 Signs Mounted on the Side of Structures

In addition to aesthetic reasons, signs attached to the side of bridge superstructures and retaining walls are difficult to inspect and maintain due to lack of access. Attachment accessories are susceptible to deterioration from de-icing chemicals, debris collection and moisture; therefore, the following guidance should be considered when detailing structure side mounted signs and related connections:

1. Limit the sign depth to a dimension equal to the bridge superstructure depth (including parapet) minus 3 inches.

2. Provide at least two point connections per supporting bracket.

3. Utilize cast-in-place anchor assemblies to attach sign supports onto new bridges and retaining walls.

4. Galvanized or stainless steel adhesive concrete masonry anchor type L may be used to attach new signs to the vertical face of an existing bridge or retaining wall. Overhead installation is not allowed. Reference Section 40.16 for applicable concrete masonry anchor requirements.

39.1.2.2 Overhead Structure Mounted Signs

Span deflections of the superstructure due to vehicle traffic are felt in overhead sign structures mounted on those bridges. The amount and duration of sign structure deflections is dependent on the stiffness of the girder and deck superstructure, the location of the sign on the bridge, and the ability of the sign structure to dampen those vibrations out; among others. These vibrations are not easily accounted for in design and are quite variable in nature. For these reasons, the practice of locating overhead sign structures onto bridges should be avoided whenever possible.

The following general guidance is given for those instances where locating a sign structure onto a bridge structure is unavoidable, which may be due to the length of the bridge, or a safety need to guide the traveling public to upcoming ramp exits or into specific lanes on the bridge.

1. Locate the sign structure support bases at pier locations.

2. Build the sign structure base off the top of the pier cap.

3. Provide set back of the upright support of the sign structure behind the back face of the parapet to preclude snagging of any vehicle making contact with the parapet.

4. Use single pole sign supports (equal balanced butterfly's) in lieu of cantilevered (with an arm on only one side of the vertical support) sign supports.

5. Consider the use of a Stockbridge type damper in the horizontal truss of these structures.

6. Do not straddle the pier leaving one support on the pier and one support off the pier in the case of skewed substructure units for full span sign bridges.

39.2 Specifications and Standards

Reference specifications for sign structures are as follows:

- AASHTO "*Standard Specifications for Structural Supports for Highway Signs, Luminaries and Traffic Signals*"
- AASHTO "*Standard Specifications for Highway Bridges, 17th Edition*"
- State of Wisconsin "*Standard Specifications for Highway and Structure Construction*"
- ASTM "*Standards of the American Society for Testing and Materials*"

Standard details for full span 4-chord galvanized steel sign bridge, design data and details for galvanized cantilever steel sign truss and footing are given on the Chapter 39 Standard Details.

Standard details for overhead sign support bases are provided in the Standard Detail Drawing (SDD) sheets of the FDM.

Standard design data and details for break-away sign supports and sign attachment are given on the A Series of the Sign Plate Manual.

39.3 Materials

Wisconsin has historically specified API Spec. 5L, grade 42 pipe as the primary material for the design of sign bridge chords and columns. However, due to recent supply shortage, ASTM A500 grades B and C, and ASTM A53 grade B types E and S round HSS or pipe (tubular shapes) are allowed as alternate materials for sign bridge truss main members (chords and columns) less than 10 inches diameter. API Spec. 5L, grade 42 remains the preferred material for single column uprights on both full span and cantilever sign bridges due to the toughness requirement to address fatigue concerns and the non-redundant nature of these structures. All plates, bars and structural angles shall be ASTM A709 grade 36. ASTM A595 grade A, A572, and A1011 have been used by manufacturers to design round, tapered steel tube members for overhead sign support arms and uprights. When tubular shapes are used for overhead sign supports, they shall conform to the sign bridge requirements. Unless noted otherwise in the contract plans, all bolted connections for sign structures shall be made with direct tension indicating (DTI) washers and meet the applicable requirements of high strength A325 bolts as stated in Section 24.2 of this manual. More details can be found in the Standard Drawings and Standard Specifications Section 641.

39.4 Design Considerations

39.4.1 Signs on Roadway

Supports for roadside signs are of three types, depending upon the size and type of the sign to be supported. For small signs, the column supports are treated timber embedded in the ground. For larger type I signs and DMS, the columns shall be galvanized steel supported on cylindrical concrete footings. Currently, all steel column supports for roadside type I signs are designed to break-away upon impact, while DMS supports are protected and designed without a break-away system.

The following design data is employed for designing ground mount or roadside sign supports:

Wind Velocity	75 mph based on the fastest mile wind speed map and its corresponding methods to find wind pressure.
Wind Components	Normal = 1.0
	Transverse = 0.0
Ice Load	3 psf
Group Loads	% of Allowable Stress
I Dead	100
II Dead + Wind	140
III Dead + Ice + (1/2 Wind)*	140
Allowable Soil Pressure	3 ksf

* Minimum Wind Load = 25 psf

Wind loading is applied to the area of sign and supporting members.

Ice loading is applied to one face of the sign and around the surface of supporting members.

39.4.2 Overhead Sign Structures

Sign structures for support of overhead signs consist of “sign bridges” and “overhead sign supports”. Sign bridges are to be either a single column cantilever or butterfly, or a space truss sign bridge supported by one or two columns at each end. For cantilever sign bridge structures, the footing is a single cylindrical shaft with wings to prevent the overturning and twisting of the structure. For space trusses having one or two steel columns on an end, the footing is composed of two cylindrical caissons connected by a concrete cross-girder. The top surface of concrete foundations for all sign bridges is to be located 3' above the highest ground line at the foundation. Occasionally, some sign bridge columns are mounted directly on top of bridge parapets, pier caps and concrete towers instead of footings.

Sign bridges also include sign support members mounted directly onto structures. Sign attachments, such as galvanized steel I-beams and/or brackets, typically are anchored to the side of the bridge superstructure. A cantilever truss attached to the side of retaining walls (without a vertical column) is also common.

Similar to sign bridges, all overhead sign supports have single galvanized steel column supported on a cylindrical caisson footing or on top of bridge elements. Cross members can be one chord (monotube), two chords without web elements, or planar truss in either cantilever or full span structure.

The following design data is employed for designing steel sign bridges and overhead sign supports.

Wind Velocity = 90 mph based on the 3-second gust wind speed map and its corresponding methods to find wind pressure.

Wind Components	Normal	Transverse
Combination 1	1.0	0.2
Combination 2	0.6	0.3

Table 39.4-1
Wind Components

Dead Load = Wt. of Sign, supporting structure, catwalk and lights.

Ice Load = 3 psf to one face of sign and around surface of members.

Group Loads	% of Allowable Stress
I Dead	100
II Dead + Wind	133
III Dead + Ice + (1/2 Wind)*	133
IV Fatigue	**

Table 39.4-2
Allowable Overstresses

* Minimum Wind Load = 25 psf

** See Fatigue section of AASHTO for fatigue loads and stress range limits.

WisDOT policy item:

Fatigue group loads application is exempt on 4-chord full span sign bridges with truss type uprights mounted on concrete footings.

Steel cantilevered sign bridge structures (4-chord structures carrying type I signage) are classified, for purposes of fatigue design, as Category 1 structures. These cantilevered support structures are designed to resist Natural Wind Gust and Truck-Induced Gust wind effects. 4-

chord cantilevered sign supports carrying type I signage are not designed for Galloping wind effects due to the substantial stiffness and satisfactory performance history in this state.

Steel cantilever sign bridge trusses are designed and fabricated from tubular shapes for chords and angle shapes for web members. Columns are made from pipe sections. The minimum thickness for the members is indicated on the steel cantilever Standard detail.

Steel full span sign bridge trusses are designed and fabricated from tubular shapes for chords and angle shapes for web members. The minimum thickness of steel web members is 3/16 inch and 0.216 inches for chord members. The connections of web members to chords are designed for bolting or shop welding to allow the contractor the option to either galvanize individual members or complete truss sections after fabrication. The upright columns are either steel pipe or tubular shape sections depending on whether they are single column or truss, see Section 39.3 for additional details. Steel base plates are used for anchor bolt support attachment.

When butt welding box sections, a back-up plate is required since the plates can only be welded from one side. The plate must be of adequate width for film to be used during weld inspection. The exposed weld is ground smooth for appearance as well as fatigue.

Aluminum sign bridges are currently not being designed for new structures. Rehabilitation and repair type work may require use of aluminum members and shall be allowed in these limited instances. The following guidelines apply to aluminum structures in the event of repair and rehabilitation type work.

Aluminum sign bridge trusses are designed and fabricated from tubular shapes shop welded together in sections. The minimum thickness of truss chords is ¼ inch and the minimum outside diameter is 4 inches. The recommended minimum ratios of “d/D” between the outside diameters “d” of the web members and “D” of chord members is 0.4. A cast aluminum base plate is required to connect the aluminum columns to the anchor bolts. AASHTO Specifications require damping or energy absorbing devices on aluminum overhead sign support structures to prevent vibrations from causing fatigue failures. Damping devices are required before and after the sign panels are erected on all aluminum sign bridges. Stock-bridge type dampers are recommended.

Install permanent signs to sign structures at the time of erection. If the signs are not available, install sign blanks to control vibration. For sign bridges, blanks are attached to a minimum of one-fourth the truss length near its center. The minimum depth of the blanks is equal to the truss depth plus 24 inches. The blanks are to be installed to project an equal dimension beyond the top and bottom chord members. Overhead sign support blanks are equal to the same sizes and at the same locations as the permanent signs. Contact BOS Structures Design Section at 608-267-2869 for further guidance on other vibration controlling methods.

Do not add catwalks to new sign bridges unless they contain DMS over traffic. Catwalks add additional cost to a structure and present a maintenance issue. They can be added if a decision is made to light the signs in the future. Design structures for a 20’-3” (2’-0” additional) vertical clearance when they are located in a continuous median freeway lighting area, for new and replacement <u>sign bridges</u> only. The sign bridge should be structurally designed to support a catwalk for those cases when the additional clearance is provided for possible future attachment. Additional accommodations for potential future lighting include providing hand holes in the uprights, rodent screens and conduits in the concrete bases.

For structures that are not located in continuous median freeway lighting areas or do not contain DMS, the additional 2 feet of structure height should not be utilized. Therefore, the design vertical clearance should be 18'-3" for new and replacement sign bridges only. No hand holes, rodent screens or conduits shall be installed on the structure in this case.

Brackets, if required, for maintenance of light units are required to support a 2'-3" wide catwalk grating and a collapsible aluminum handrail. Brackets and handrailing for type I and II signs are fabricated from aluminum sections, whereas DMS support brackets are made of galvanized steel. Catwalk grating and toe plates are fabricated from steel and shall be galvanized.

Contract plans should note (under the general notes) if hand holes are required on one or both uprights of the sign bridge.

Overhead sign supports are typically not lit, nor do they require sign maintenance. Therefore, do not detail a catwalk on this type of structure. Also do not detail hand holes, rodent screens or conduits unless the structure is designed to carry an LED changeable message sign.

Design of all Sign Bridge structures should reflect some provision for the possibility of adding signs in the future (additional sign area). Consideration should include the number of lanes, possible widening of roadway into the median or shoulder areas, and use of diagrammatic signs to name a few. The truss design should reflect sizing the chords for maximum force at the center of the span. The design of the upright and truss webs should allow for signs being placed (say sometime in the future) more skewed to one side than the other. Uprights should be selected the same size (outside diameter x thickness) for each side and the design shall reflect different lengths on either side as required by site conditions.

The design sign area and maximum sign depth dimensions for type I and II signs shall be explicitly listed with the design data in the contract plans. Use 3 psf dead load for these types of signs. Provide manufacturer overall DMS dimensions in the plans along with the total weight of the signs. Other loads such as Catwalks, lights and associated attachments must also be included in the overall design data in the contract plans.

The following guidance is recommended for estimating design sign areas.

1. Type I and II signs on full span 4-chord sign bridges, design sign area equals the largest value resulting from the four requirements below:

    a. Total actual sign area.

    b. Two (2) times the controlling upright tributary sign area. Tributary area is computed based on the application of the lever rule on a simply supported truss.

c. Twelve (12) times the number of lanes times the maximum sign depth. The number of lanes is defined as the clear roadway width (including median and shoulders) divided by 12 and rounding down to the nearest whole number.

d. Maximum sign depth times 60% of the span length (center to center of upright).

For design purposes, the standard sign depth shall be limited between 12'-0" and 16'-0". Therefore, vertical clearance and upright lengths are to be sized with sign depth not less than 12'-0", unless requested otherwise in the structure survey report. Mega projects with series of sign bridges may deviate from the above requirements provided that coordination is made with the BOS Structures Design Section.

2. Type I and II maximum design sign area for galvanized steel cantilever sign truss is detailed in Standard 39.10. Sizing the upright length and vertical clearance with 12'-0" sign depth is recommended for future accommodation.

3. DMS sign bridges should be designed with the actual sign dimensions in addition to those of type I and II signs and catwalk as applied.

4. Overhead sign supports are generally designed with the actual sign dimensions and locations. Exception to the approach may be granted to structures with anticipated change in signage.

39.5 Structure Selection Guidelines

Sign structures are composed of “sign bridges” and “overhead sign supports”. Either type of sign structure can be configured to be a cantilever sign structure (one upright to arm) or a full-span sign structure (two uprights, one on each end of the span). Roadside sign supports are an exception to the above naming convention.

“Sign Bridges” generally carry type I and II signs, and occasionally DMS. These are large sign structures with sign depths ranging from 5’-0” or less to 18’-0” in the case of large diagrammatic signs. Total sign areas accommodated are up to 264 sq. ft. on cantilever sign bridges. Total sign areas accommodated on full span sign bridges range from 250 to over 1000 sq. ft. of sign area. These ranges are an approximate guide only. Most sign bridges generally have truss members consisting of four round chords and angle web members supporting type I and II signs on the span or arm (although some three chord structures have been used for full span sign bridges). Uprights are comprised of one pole for a cantilever sign bridge. Full span sign bridge uprights usually consist of two poles joined by angle web members at each end of the span (although single pole uprights have been used on three chord full span sign bridges). “Sign Bridges” are designed by the Bureau of Structures or a consultant. Structure contract plans provide full details that a fabricator can construct the sign bridge from. Standard details for the 4-chord sign bridge are associated with this Chapter of the Bridge Manual which requires a design for each sign bridge structure including foundations. These details are used for type I and II sign applications only.

Sign bridges carrying variable message signs require special consideration. Special concerns include:

1. Weight of the sign panel.

2. Width and weight of catwalk.

3. Consideration of wind effects unique to these signs.

4. Modification to brackets used. All catwalk and sign connection brackets are to be made with friction type connection and high strength A325 bolts with DTI washers.

Wisconsin recommends the use of the Minnesota 4-chord configuration for sign bridges carrying DMS, providing that the designer check the design of each member and connections conforming to the latest AASHTO Standard specifications requirements.

“Overhead Sign Supports” are smaller sign structures carrying both type II (smaller) directional signs, limited amounts of type I signs and small LED or changeable message signs. Type II sign depths have ranged from 3’-0” to 4’-0” deep for traffic directional signs, and up to 10’-0” for small information type I signs. When a sign is larger than 10’-0” deep, the structure is to be designed as a sign bridge. Cantilever overhead sign supports accommodate up to 45 sq. ft. of sign area. Total sign areas accommodated on full span overhead sign supports range up to 300 sq. ft. These ranges are again an approximate guide and can be more or less depending on variables such as span length, location of the sign with respect to the upright(s), the height of the upright(s), etc. Uprights are comprised of one pole (uniform round or tapered pipe) for either the cantilever or full span overhead sign support. Arms on cantilevers or the span on a full span overhead sign support are either one chord (uniform round or tapered pipe), or two chords with or without angle web members (planar truss) depending on the span length and sign depth. Due to the variability of factors that can influence the selection of structure type, designers are encouraged to contact BOS Structures Design Section for further assistance when sign areas fall outside of the above limits, or structural geometry is in question. "Overhead Sign Supports" are normally bid by the contractor and designed by a fabricator or by another party for a fabricator to construct. These structures usually have the least plan detail associated with them and are normally depicted in the Construction Detail portion of the state contract plans. However, it is recommended that plan development for projects with multiple structures, such as major or mega projects, be prepared by structural engineers and placed in section 8 of the contract plans along with the sign bridge plans. A concrete footing base design is required to be shown in the contract plans for overhead sign supports. See the WisDOT FDM Procedures 11-55-20 and 15-1-20 for more information on Overhead Sign Supports.

39.6 Geotechnical Guidelines

Several potential problems concerning the required subsurface exploration for foundations of sign structures exist. These include:

- The development and location of these structures are not typically known during the preliminary design stage, when the majority of subsurface exploration occurs. This creates the potential for multiple drilling mobilizations to the project.
- Sometimes these structures are located in areas of proposed fill soils. The source and characteristics of this fill soil is unknown at the time of design.
- The unknowns associated with these structures in the scoping/early design stages complicate the consultant contracting process. How much investigation should be scoped in the consultant design contract?

Currently, all sign structure foundations are completely designed and detailed in the project plans. Sign-related design information can be found in the Facilities Development Manual (FDM) or Bridge Manual as described in the following sections.

39.6.1 Sign Bridges

WisDOT has created a standard foundation design for cantilever sign bridges carrying Type I signs. This standard foundation is presented on Standard 39.12 of the Bridge Manual Standard Details. The wings on this single shaft footing are used to help resist torsion. If the cantilever sign bridge, carrying Type I signs, exceeds the criteria/limitations (shown on Standard 39.10), the standard foundation shall not be utilized, and an individual foundation must be fully designed. This customized design will involve~~s~~ determining the subsurface conditions as described in section 39.6.3.

Foundations supporting all full span sign bridges are custom designed. They generally have two cylindrical shafts connected by a concrete cross-girder below each vertical upright. WisDOT has no standard details for the foundations for these structures.

WisDOT policy item:

The length of a cast-in-place shaft foundation shall be limited to 20'-0". Deviation from this policy item may be allowed provided coordination is made with BOS Structures Design Section.

39.6.2 Overhead Sign Supports

Overhead sign supports are described in Sections 11-55-20 and 15-1-20 of the FDM. In addition, Section 641 of the Standard Specifications outlines the design/construction aspects of these structures.

If these structures meet several criteria/limitations that are listed on the SDD's, the designer can use WisDOT-developed standard foundations for them. The designer can then insert the proper SDD sheet into the plans. SDD sheets exist for cantilever overhead sign supports.

These single shaft bases for cantilever overhead sign supports vary in depth and range from 24” to 42” in diameter. Another SDD sheet applies to full-span overhead sign supports and is 36” in diameter. The standard foundations in these SDD sheets were designed using slightly conservative soil design parameters. If the design criteria for these standard designs are not met, the SDD sheets cannot be used and the structure foundation must be fully designed and the unique details supplied in the construction details portion of the contract plans. This involves determining the subsurface conditions as described in the following section.

39.6.3 Subsurface Investigation and Information

No subsurface investigation/information is necessary for any of the sign structures that meet the limitations for allowing the use of WisDOT standard foundations. Appropriate subsurface information is necessary for any of these structures that require custom designs.

There may be several methods to obtain the necessary subsurface soil properties to allow for a custom design of foundations, as described below:

- In areas of fill soils, the borrow material may be unknown. The designer should use their best judgment as to what the imported soils will be. Standard compaction of this material can be assumed. Conservative soil design parameters are encouraged.

- The designer may have a thorough knowledge of the general soil conditions and properties at the site and can reasonably estimate soil design parameters.

- The designer may be able to use information from nearby borings. Judgment is needed to determine if the conditions present in an adjacent boring(s) are representative of those of the site in question.

- If the designer cannot reasonably characterize the subsurface conditions by the above methods, a soil boring and Geotechnical report (Site Investigation Report) should be completed. Necessary soil design information includes soil unit weights, cohesions, phi-angles and location of water table.

Designers, both internal and consultant, should also be aware of the potential of high bedrock, rock fills and the possible conflict with utilities and utility trenches.

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40.5 Deck Overlays

If the bridge is a candidate for replacement or a new deck, serviceability may be extended 3 to 7 years by patching and/or overlaying the deck with only a 1-1/2" minimum thickness asphaltic mat on lightly traveled roadways. Experience indicates the asphalt tends to slow down the rate of deterioration while providing a smooth riding surface. However, these decks must be watched closely for shear or punching shear failures as the deck surface problems are concealed.

For applications where the deck is structurally sound and service life is to be extended there are other methods to use. A polymer modified asphaltic overlay may be used to increase deck service life by approximately 15 years. If the concrete deck remains structurally sound, it may be practical to remove the existing overlay and place a new overlay before replacing the deck.

A 1-1/2" concrete overlay is expected to extend the service life of a bridge deck for 15 to 20 years. On delaminated but structurally sound decks a concrete overlay is often the only alternative to deck replacement. Prior to placing the concrete overlay, a minimum of 1" of existing deck surface should be removed. On all bridges low slump Grade E concrete is the specified standard with close inspection of concrete consolidation and curing. If the concrete deck remains structurally sound; it may be practical to remove the existing overlay and place a second deck overlay before replacing the entire deck. After the concrete overlay is placed, it is very important to seal all the deck cracks. Experience shows that salt water passes thru these cracks and causes deterioration of the underlying deck.

On deck overlays preparation of the deck is an important issue after removal of the top surface. Check the latest Special Provisions and/or specifications for the method of payment for Deck Preparation where there are asphalt patches or unsound concrete.

Micro-silica concretes have been effectively used as an alternate type of concrete overlay. It provides excellent resistance to chloride penetration due to its low permeability. Micro-silica modified concrete overlays appear very promising; however, they are still under experimental evaluation. Latex overlays when used in Wisconsin have higher costs without noticeable improved performance.

Ready mixed Grade E concrete with superplasticizer and fiber mesh have been tried and do not perform any better than site mixed concrete produced in a truck mounted mobile mixer.

Bridges with Inventory Ratings less than HS10 with an overlay shall not be considered for concrete overlays, unless approved by the Bureau of Structures Design Section. Bridges reconstructed with overlays shall have their new Iinventory and Operating Ratings shown on the bridge rehabilitation plans. Verify the desired transverse cross slope with the Regions as they may want to use current standards.

40.5.1 Guidelines for Bridge Deck Overlays

As a structure ages, rehabilitation is a necessary part of insuring a level of acceptable serviceability. Overlays can be used to extend the service lives of bridge decks that have surface deficiencies. Guidelines for determining if an overlay should be used are:

- The structure is capable of carrying the overlay deadload;
- The deck and superstructure are structurally sound;
- The desired service life can be achieved with the considered overlay and existing structure;
- The selected option is cost effective based on the structure life.

40.5.2 Deck Overlay Methods

An AC Overlay or Polymer Modified Asphaltic Overlay should not be considered on a bridge deck which has a longitudinal grade in excess of four percent or an extensive amount of stopping and starting traffic. All full depth repairs shall be made with PC concrete.

Guidelines for determining the type of deck overlay method to achieve the desired extended service life are:

AC Overlay (ACO): 5 years average life expectancy

- The minimum asphaltic overlay thickness is 1-1/2".
- The grade change due to overlay thickness can be accommodated at minimal cost.
- Deck or bridge replacement is programmed within 7 years.
- Raising of floor drains or joints is not required.
- Spalls can be patched with AC or PC concrete with minimal surface preparation.

Polymer Modified Asphaltic Overlay: 15 to 20 years life expectancy

- This product may be used as an experimental alternate to LSCO given below. CAUTION – Core tests have shown the permeability of this product is dependent on the aggregate. Limestone should not be used.

Polymer Overlay: 10 to 15 years life expectancy

- A 1/4-inch thick, two layer system comprised of a two-component polymer in conjunction with natural or synthetic aggregates. Use 5 psf for dead load, DW.
- Works well to seal decks and/or provide traction.

40.8 Widenings

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

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

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

For multi-columned piers, consider the cap connection to the existing cap as pinned and design the new portion to the latest LRFD criteria. The new column(s) are not required to meet AASHTO 3.6.5 (600 kip loading) as a widening is considered rehabilitation. Abutments shall be widened to current LRFD criteria as well as the Standard for Abutment Widening.

For foundation support, use the most current method available.

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

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

40.9 Superstructure Replacements/Moved Girders (with Widening)

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

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

Evaluate the existing piers using current LRFD criteria. If an existing multi-columned pier has 3 or more columns, the 600 kip vehicular impact loading need not be considered if the pier is adjacent to a roadway with a design speed ≤ 40 mph. If the design speed is 45 mph or 50 mph, the 600 kip vehicular impact loading need not be considered if a minimum of “vehicle protection” is provided as per FDM 11-35-1. For design speeds > 50 mph, all criteria as per 13.4.10 must be met.

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

The superstructure shall be designed to current LRFD criteria.

When using “Type S” anchors to anchor bolts or studs, the bolt, nut, and washer or the stud as detailed on the plans is included in the bid item “Masonry Anchors Type S _-Inch”.

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

Adhesive “Type L” anchors located in uncracked concrete:

> MASONRY ANCHORS TYPE L NO. X BARS. EMBED XX” IN CONCRETE. *(Illustrative only, values must be calculated depending on the specific situation).*

Adhesive “Type L” anchors located in cracked concrete:

> MASONRY ANCHORS TYPE L NO. X BARS. EMBED XX” IN CONCRETE. ANCHORS SHALL BE APPROVED FOR USE IN CRACKED CONCRETE. *(Illustrative only, values must be calculated depending on the specific situation).*

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

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

40.17 Plan Details

1. Excavation for Structures on Overlays

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

   > Any excavation required to complete the overlay or paving block at the abutments is to be considered incidental to the bid item "(insert applicable bid item)".

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

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

3. On structure deck overlay projects:

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

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

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

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

5. Total Estimated Quantities

   The Region should provide the designer with a Rehabiliation Structure Survey Report that provides a complete description of the rehabilitation and estimated quantities. Contact the Region for clarifications on the scope of work.

Additional items:

- Provide deck survey outlining areas of distress (if available). These plans will serve as documentation for future rehabilitations.

- Distressed areas should be representative of the surveyed areas of distress. Actual repairs will likely be larger than the reported values while removing all unsound materials.

- Provide Preparation Deck Type 1 & 2 and Full-Depth Repair estimates for areas of distress.

- Coordinate asphaltic materials with the Region and roadway designers.

See Chapter 6-Plan Preparation and Chapter 40 Standards for additional guidance.

40.18 Retrofit of Steel Bridges

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

40.18.1 Flexible Connections

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

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

40.18.2 Rigid Connections

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

Other partial constraint details are:

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

The solution is to create spaces large enough (approximately 1/4” or more) for more material to flex thus reducing the concentration of stress. For gusset connection plates provide a larger gap than 1/4” and no intersecting welds. For existing conditions it may be necessary to drill holes at high stress concentrations. For new conditions it would be better to design a rigid connection and attach to the flange rather than the web. For certain situations a fillet weld should be used over a partial penetration weld to allow slight movement.

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45.4.2 Posting Signage

Current WisDOT policy is to post State bridges for only one tonnage capacity. Bridges which cannot carry the maximum weight for the vehicles described in Section 45.4.1 using Operating Rating criteria are posted with one of the standard signs, shown in Figure 45.4-5 showing the bridge capacity for the governing vehicle, which 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 45.4-6 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 Engineer.

WEIGHT LIMIT 10 TONS	BRIDGE CLOSED

Figure 45.4-5
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 45.4-6
Historic Load Posting Signs

45.5 Material Strengths and 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.

45.5.1 Reinforcing Steel

The allowable unit stresses and yield strengths for reinforcing steel can be found in Table 45.3-1. 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 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.