Table of Contents

4.1 Introduction

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

Desirable bridge aesthetics do not necessarily need to cost much, if any, additional money. Aesthetic enhancements can be made in a number of ways. Primary features such as structure type and shape have the most influence on appearance, with color and texture playing secondary roles. Formliners, especially when used in conjunction with a multi-colored stain, are more expensive than one or two single color stains on smooth concrete, and have on a number of occasions not fit the context of the project. It is the responsibility of the design team to identify aesthetic treatments that are consistent with the environment and goals of the project, are maintainable over the life of the structures, and are cost effective. See 4.5 for current policy regarding structure aesthetics.

While initial cost for aesthetic enhancements is a concern, it has become apparent that maintenance costs can be considerably more than initial costs. Stain, which acts more like paint, must be periodically redone. Such reapplication oftentimes requires lane closures which are both an undesirable inconvenience to users and come with a significant cost associated with maintenance-of-traffic.

4.2 General Aesthetic Guidelines

Primary features – in relative order of importance:

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

Secondary features – in relative order of importance:

- Color
- Pattern and texture
- Ornamentation

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

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

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

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

Refer to the WisDOT *Traffic Engineering, Operations and Safety Manual* section 2-1-60 for guidance on community sensitive design signing.

4.4 Secondary Features

Color

Color can have a strong visual effect, either positive or negative. Using earth toned colors versus vivid colors is preferred. More neutral colors tend to blend in more with the surroundings. Also, over time earth tones will weather less and not appear as dingy or faded. A bright yellow, for example, will begin to appear dull and dirty soon after application. Avoid red as this color is not UV tolerant and will fade. Concrete stain behaves more like paint and is susceptible to fading and peeling, requiring re-application to avoid an unsightly structure. Stained concrete in need of maintenance looks worse than concrete that was originally left unstained.

Using a maximum of two colors will lend itself to the desired outcome of a clean appearance. On larger structures it may be desirable to use two colors for everything other than the girders, which may be a third color. Remember that plain concrete is a color, too. It should be utilized as much as possible (especially on smaller surfaces) to reduce initial cost and, especially, future maintenance costs.

Utilizing a ribbed, or broken ribbed pattern on a large expanse of plain concrete can give the appearance of color as the patterned section will appear darker than the adjacent plain concrete. This is a good way to add 'color' without the future maintenance costs associated with actual stain reapplication.

As much as possible, *AMS Standard Color Numbers* should be used for color selection. A few colors are given in Chapter 9 – Materials, but others may be used. STSP's should be used as is for staining and multi-colored staining. Specific colors, areas to be applied, etc. should be referenced on the plan sheets.

Pattern and Texture

See 4.5 for current policy regarding structure aesthetics, including patterns and texture.

Large expanses of flat concrete, even if colored, are usually not desirable.

Most bridges are seen from below by people traveling at higher rates of speed. Detail smaller than 4-inches is difficult to discern. The general shape, and perhaps color, will have a greater visual effect than the pattern and/or texture. Sometimes texture is used to represent a building material that wasn't used for the construction of the structure, as would be the case of rock form liner. <u>While a rock appearance might be appropriate for a smaller bridge over a stream in a small town, it seldom fits the context of a grade separation over a highway or busy urban interchange.</u> Modern bridges should, for the most part, look like they are built out of modern materials appropriate to the current time. Texture consisting of random or ordered geometric forms is generally more preferred over simulating other materials.

On MSE retaining walls it is desirable to keep logos or depictions within a given panel. Matching lines across panels, especially horizontal lines susceptible to differential panel settlement, is difficult. Rock texturing is unconvincing as real stone due to panel joints. A random geometric pattern is a good way to give relief to a wall.

Repetition in pattern rather than an assembly of various patterns or details is more cost effective. For effects that are meant to appear random (e.g. rock), care must be taken in order for the pattern repetition to not appear noticeable.

At all locations on a structure (abutment wings and piers, MSE walls, etc.), form details should be terminated 1'-0" below low water or ground elevations where they will not be visible. See the Standard for Formliner Details.

Designers are cautioned about introducing textures and relief on the inside faces of vehicle barriers. The degree of relief and texture can influence the vehicle response during a crash. See Chapter 30 – Railings for further guidance.

Ornamentation

If signs or medallions are necessary, refer to section 2-1-60 of the *Traffic Engineering, Operations and Safety Manual*.

Regarding ornamentation in general, more is seldom better.

"In bridge building… to overload a structure or any part thereof with ornaments… would be to suppress or disguise the main members and to exhibit an unbecoming wastefulness. The plain or elaborate character of an entire structure must not be contradicted by any of its parts."

- J.B. Johnson, 1912

4.6 Level of Aesthetics

The Regional Office should establish one of the following levels of aesthetics and indicate it on the Structure Survey Report. This will help the structural designer decide what level of effort and possible types of aesthetics treatments to consider. If Level 2 or greater is indicated, the Regional Office personnel or consultant must suggest particular requirements such as railing type, pier shape, special form liners, color, etc. in the comments area of the Structure Survey Report. Most Regions/municipalities prefer to leave anti-graffiti coating off of structures and would rather re-stain, as this is easier than trying to clean the graffiti.

Aesthetic treatments should be agreed upon prior to completion of preliminary plans in order for the final design to proceed efficiently. These details would be developed through the aesthetic process.

1. Level One: A general structure designed with standard structure details. This would apply in rural areas and urban areas with industrial development.

2. Level Two: Consists of cosmetic improvements to conventional Department structure types, such as the use of color stains/paints, texturing surfaces, modifications to fascia walls and beams or more pleasing shapes for columns. This would apply where there needs to be less visual impact from roadway structures.

3. Level Three: Emphasize full integration of efficiency, economy and elegance in structure components and the structure as a whole. Consider structure systems that are pleasing such as shaped piers and smooth superstructure lines. These structures would need to be in harmony with the surrounding buildings and/or the existing landscape.

4. Level Four: Provide overall aesthetics at the site with the structure incorporating level three requirements. The structure would need to blend with the surrounding terrain and landscaping treatment would be required to complete the appearance.

Note: The above text was left in this chapter, but will likely be modified or removed in future editions of this Manual. See 4.5 for current policy regarding CSS and levels of aesthetics.

4.7 Accent Lighting for Significant Bridges

The Wisconsin DOT will consider as part of an improvement project accent lighting for significant urban bridges with a clear span length of 450 feet and greater. The lighting would accent significant components above the driving surface such as an arch, truss, or a cable stayed superstructure. This lighting would enhance the noteworthy structure components of these significant bridges. The *Traffic Engineering, Operations and Safety Manual* (TEOpS) and the *Program Management Manual* (PMM) have respective guidance of maintenance and cost share policy.

The following structures would fall into this definition of significant urban bridges:

"Name"	Region	County	Feature On	Feature Under	Year Built	Border
Tower Drive	NE	Brown	IH 43	Fox River	1979	
Praire du Chien	SW	Crawford	USH 18-STH 60	Mississippi River	1974	X
Blatnik	NW	Douglas	IH 535-USH 53	St Louis Bay	1961	X
Bong	NW	Douglas	USH 2	St Louis River	1983	X
Cass Arch	SW	La Crosse	USH 14 EB	Mississippi River	2004	X
Cass Truss	SW	La Crosse	USH 14 WB	Mississippi River	1940	X
Hoan Bridge	SE	Milwaukee	IH 794 WB-Lake Freeway	Milwaukee River	1974	
Dubuque (Iowa)	SW	Grant	USH 61-USH 151	Mississippi River	1982	X
Stillwater	NW	St Croix	TH 36	St Croix River	New	X

Table 4.4-1
Accent Lighting for Significant Bridges

Table of Contents

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.

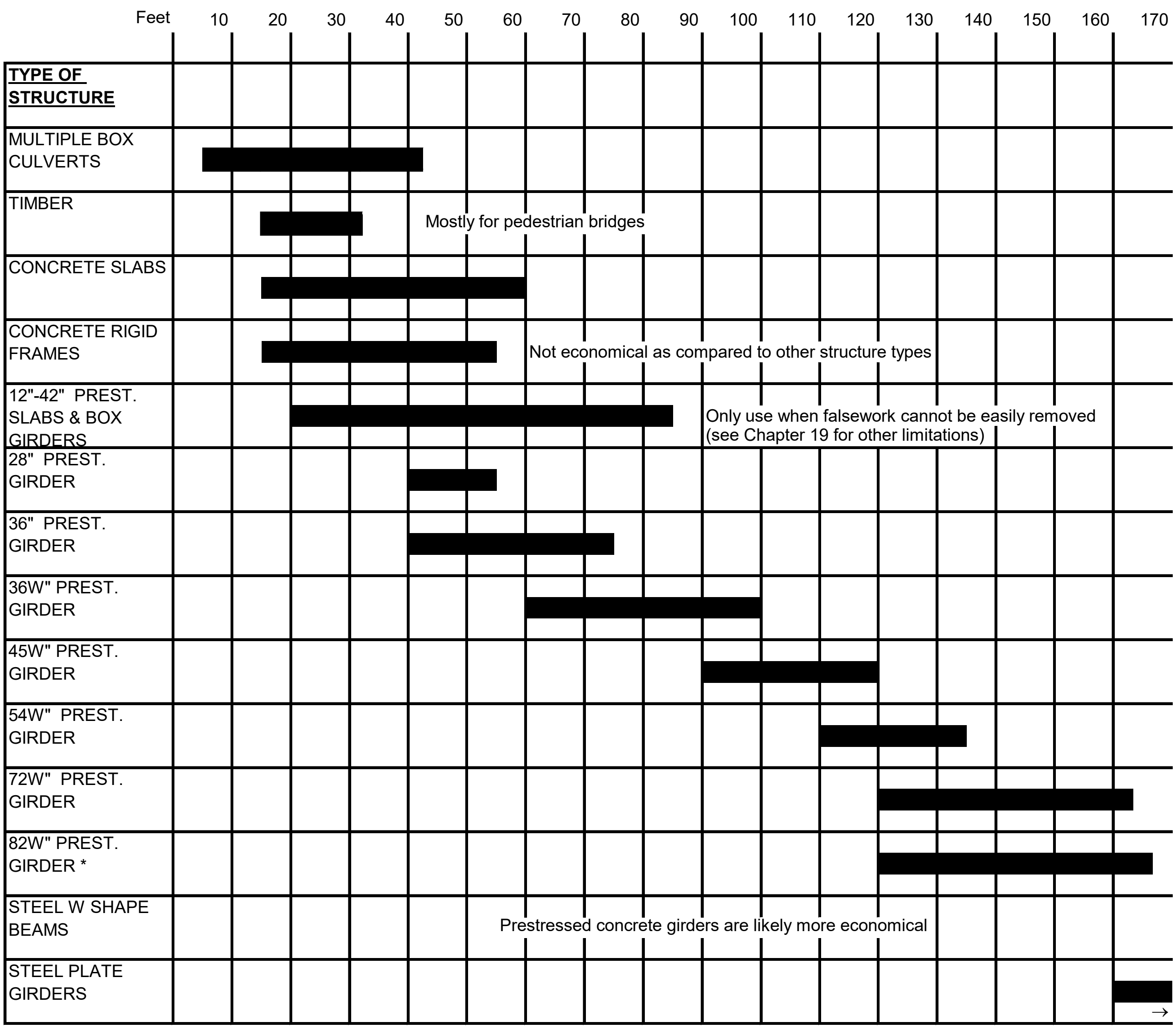
In these reports prestressed girder costs are grouped together because there is a small cost difference between girder sizes. Refer to unit costs. Concrete slab costs are also grouped together for this reason.

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 Facilities Development Manual (FDM) 19-5-5, should be the primary tools for structure project cost estimations. Information in this chapter can be used as a supplemental tool.

5.2 Economic Span Lengths



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

Figure 5.2-1
Economic Span Lengths

5.3 Contract Unit Bid Prices

Item No.	Bid Item	Unit	Cost
502.3100	Expansion Device (structure) (LS)	LF	$230.37
502.3110.S	Expansion Device Modular (structure) (LS) (2017)	LF	$1401.52
SPV.0105	Expansion Device Modular LRFD (structure) (LS) (2017)	LF	$1947.75

Table 5.3-1
Contract Unit Bid Prices for Structures - 2018

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

5.4 Bid Letting Cost Data

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

The square foot costs include all items shown on the structure plan except removing old structure. Costs also include a proportionate share of the project's mobilization, as well as structural approach slab costs, if applicable. However, square footage does not include the structural approach slabs, and is based on the length of the bridge from abutment to abutment. (It is realized that this yields a slightly higher square footage bridge cost for those bridges with structural approach slabs.)

5.4.1 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 5.4-1
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	29	409,929	44,335,036	64.66	108.15
Reinf. Conc. Slabs (All but A5)	2	15,072	1,739,440	47.68	115.41
Steel Plate Girders	3	85,715	15,669,789	114.08	182.81
Steel I-Beams	1	2,078	596,712	82.99	287.16
Trapezoidal Steel Box Girders	1	59,128	9,007,289	121.00	152.34
Pedestrian Bridges	3	35,591	7,436,429	--	208.94

Table 5.4-2
Grade Separation Structures

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

Table 5.4-3
Box Culverts

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

Table 5.4-4
Retaining Walls

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

Table 5.4-5
Noise Walls

5.4.2 2015 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	22	338,229	41,220,154	60.96	121.87
Reinf. Conc. Slabs (Flat)	26	47,766	7,151,136	62.77	149.71
Reinf. Conc. Slabs (Haunched)	6	27,967	3,517,913	57.49	125.79
Buried Slab Bridges	1	2,610	401,000	43.74	153.64
Pre-Fab Pedestrian Bridges	3	29,304	3,440,091	--	117.39

Table 5.4-6
Stream Crossing Structures

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

Table 5.4-7
Grade Separation Structures

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

Table 5.4-8
Box Culverts

Retaining Wall Type	No. of Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Block Walls	11	22,353	1,594,171	71.32
MSE Panel Walls	51	315,440	28,038,238	88.89
MSE Panel Walls w/Integral Barrier	4	14,330	1,098,649	76.67
Concrete Walls	2	6,850	712,085	103.96
Wire Faced MSE Walls	3	10,345	1,501,948	145.19
Wire Faced MSE Walls w/ Precast Conc. Wall Panels	12	50,670	10,195,161	201.21
Secant Pile Walls	1	5,796.50	1,075,785	185.59
Soldier Pile Walls	6	37,498	6,037,788	161.02
Steel Sheet Pile Walls	6	11,319	668,227	59.04

Table 5.4-9
Retaining Walls

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

Table 5.4-10
Sign Structures

5.4.3 2016 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	19	199,367	26,157,660	57.97	131.20
Reinf. Conc. Slabs (Flat)	36	72,066	10,985,072	63.40	152.43
Reinf. Conc. Slabs (Haunched)	5	22,144	2,469,770	50.63	111.53
Prestressed Box Girders	3	4,550	773,098	80.85	169.91

Table 5.4-11
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	25	343,165	40,412,805	60.62	117.76
Reinf. Conc. Slabs (Haunched)	5	33,268	4,609,286	59.21	138.55
Steel Plate Girders	3	127,080	18,691,714	90.78	147.09
Pedestrian Bridges	1	4,049	846,735	91.35	209.13

Table 5.4-12
Grade Separation Structures

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

Table 5.4-13
Box Culverts

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

Table 5.4-14
Retaining Walls

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

Table 5.4-15
Sign Structures

5.4.4 2017 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super Only Cost per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	24	238,956	33,970,344.86	60.05	142.16
Reinf. Conc. Slabs (Flat)	44	69,095	11,063,299.53	57.75	160.12
Reinf. Conc. Slabs (Haunched)	8	48,434	6,759,897.64	55.41	139.57
Prestressed Box Girders	2	2,530	691,474.35	117.93	273.31

Table 5.4-16
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	28	302,672	37,247,580.94	52.67	123.10
Reinf. Conc. Slabs (Flat)	25	58,076	9,561,823.06	42.14	164.64
Reinf. Conc. Slabs (Haunched)	6	49,160	9,444,012.75	43.73	192.11
Steel Plate Girders	0	--	--	--	--
Pedestrian Bridges	2	12,864	2,141,133.01	53.53	166.44

Table 5.4-17
Grade Separation Structures

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

Table 5.4-18
Box Culverts

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

Table 5.4-19
Retaining Walls

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

Table 5.4-20
Sign Structures

5.4.5 2018 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	45	276,820	45,260,979	66.45	163.50
Reinf. Conc. Slabs (Flat)	49	72,180	12,259,362	68.04	169.85
Reinf. Conc. Slabs (Haunched)	8	34,732	6,437,911	70.04	185.36
Prestressed Box Girder	1	1,864	419,175	113.39	224.88

Table 5.4-21
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	52	727,872	124,665,613	59.90	171.30
Reinf. Conc. Slabs (Haunched)	6	56,580	10,858.579	57.14	191.92
Steel Plate Girders	0	--	--	--	--
Trapezoidal Steel Box Girders	0	--	--	--	--

Table 5.4-22
Grade Separation Structures

Box Culvert Type	No. of Culverts	Cost per Lin. Ft.
Single Cell	13	1,948
Twin Cell	6	2,941
Three Cell	1	6,354

Table 5.4-23
Box Culverts

Bridge Type	Cost
Twin Pipe Culvert	2,292 Lin. Ft.

Table 5.4-24
Miscellaneous Bridges

Retaining Wall Type	No. of Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
CIP Cantilever	0	--	--	--
CIP Facing (MSE)	0	--	--	--
MSE Block Walls	3	4,693	567,547	120.93
MSE Panel Walls	49	378,371	44,841,726	118.51
Modular Walls	3	2,402	204,002	84.93
Precast Panel and Wire Faced	1	5,945	948,347	159.53
Soldier Pile Walls	4	8,531	1,570,107	184.05
Steel Sheet Pile Walls	2	16,620	1,639,380	98.64

Table 5.4-25
Retaining Walls

Sign Structure Type		No. of Structures	Total Lineal Ft. of Arm	Total Costs	Cost per Lin. Ft.
Butterfly (1-Sign)	Conc. Col.	6	118	273,756	2,319.97
	1-Steel Col.	0	--	--	--
Butterfly (2-Signs)	Conc. Col.	5	88	277,787	3,156.67
	1-Steel Col.	4	73	326,652	4,474.68
Cantilever	Conc. Col	8	234	588,676	2,515.71
	1-Steel Col	32	850.83	1,380,710	1,622.78
Cantilever Full Span	Conc. Col.	16	1267	2,909,973	2,296.74
	1-Steel Col.	2	184.2	279,115	1,515.28
	2-Steel Col.	17	1469	2,236,464	1,522.44
Full Span	1-Steel Col.	10	675.5	513,623	760.36
	2-Steel Col.	0	--	--	--

Table 5.4-26
Sign Structures

Table of Contents

6.1 Approvals, Distribution and Work Flow

Production of Structural Plans

Regional Office	Prepare Structure Survey Report.
Geotechnical Section (Bur. of Tech. Services)	Make site investigation and prepare Site Investigation Report. See 6.2.1 for exceptions.
Structures Development Sect. (BOS)	Record Structure Survey Report.
Structures Design Section (BOS)	Determine type of structure.
	Perform hydraulic analysis if required.
	Check roadway geometrics and vertical clearance.
	Review Site Investigation Report and determine foundation requirements. Develop scour computations for bridges and record scour code on the preliminary plans.
	Draft preliminary plan layout of structure.
	Send copies of preliminary plans to Regional Office.
	If Federal aid funding is involved, send copies of preliminary plans to the Federal Highway Administration for major, moveable, and unusual bridges.
	If a waterbody that qualifies as a “navigable water of the United States” is crossed, a Permit drawing to construct the bridge is sent to the Coast Guard. If FHWA determines that a Coast Guard permit is needed, send a Permit drawing to the Coast Guard. If Federal aid is involved, preliminary plans are sent to

	the Federal Highway Administration for approval.
	Review Regional Office comments and other agency comments, modify preliminary plans as necessary.
	Review and record project for final structural plan preparation.
Structures Design Units (BOS)	Prior to starting project, Designer contacts Regional Office to verify preliminary structure geometry, alignment, width and the presence of utilities.
	Prepare and complete plans, specs and estimates for the specified structure.
	Give completed job to the Supervisor of Structures Design Unit.
Supervisor, Structures Design Unit (BOS)	Review plans, specs and estimates.
	Send copies of final structural plans and special provisions to Regional Offices.
	Sign lead structural plan sheet.
	Deliver final structural plans and special provisions to the Bureau of Project Development.
Bur. of Project Development	Prepare final approved structural plans for pre-contract administration.

See Facilities Development Manual (FDM) Section 20-50-5 for information on determining whether a bridge crossing falls under the Coast Guard's jurisdiction.

6.2 Preliminary Plans

6.2.1 Structure Survey Report

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

Under the "Plans and Projects" heading:

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

The front of the form lists the supplemental information to be included with the report.

6.2.1.1 BOS-Designed Structures

When preparing the Structure Survey Report, the region or consultant roadway designers will make their best estimate of structure type and location of substructure units. The completed Structure Survey Report with the locations of the substructure units and all required attachments and supporting information will then be submitted to the Bureau of Structures via e-submit (as "BOS Design") and also to the Geotechnical Section, through the Regional Soils Engineer. This submittal will take place a minimum of 18 months in advance of the earliest PS&E due date shown on the Structure Survey Report. The Geotechnical Section is responsible for scheduling and conducting the necessary soil borings. The Bureau of Structures and the Geotechnical Section will coordinate activities to deliver the completed structure plans on schedule.

When a geotechnical consultant is performing the subsurface exploration, the work typically proceeds after the preliminary plans have been assembled by the Bureau of Structures. Under some circumstances, it may be expected that the geotechnical information gathered will be included in the Structure Survey Report in advance of the development of the preliminary plans. In the case of the Geotechnical Section performing the subsurface exploration, the geotechnical work will proceed after the preliminary plans have been assembled by the Bureau of Structures.

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

Under this process, the scheduling of geotechnical work is coordinated with the Bureau of Structures toward completion of the bridge plans by the final plan due date. If other geotechnical work is required for the project, the Project Manager should coordinate with the

Regional Soils Engineer and the Geotechnical Section to promote efficiency of field drilling operations.

If the preliminary plans are required more than one year in advance of the final plan due date due to the unique needs of the project, the Project Manager should discuss this situation with the Bureau of Structures Design Supervisor prior to submitting the Structure Survey Report.

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

6.2.1.2 Consultant-Designed Structures

When preparing the Structure Survey Report, the region or consultant roadway designer’s responsibility for submitting the Structure Survey Report depends on their involvement with the design of the structure and the soils investigation.

If the preliminary bridge plans are required more than one year in advance of the final plan due to the unique needs of the project, the Project Manager should discuss this situation with the consultant.

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

6.2.2 Preliminary Layout

6.2.2.1 General

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

The drawings for preliminary layouts are on sheets having an overall width of 11 inches and an overall length of 17 inches and should be placed within the current sheet border under the #8 tab.

6.2.2.2 Basic Considerations

The following criteria are used for the preparation of preliminary plans.

1. Selection of Structure Type. Refer to Chapter 17 - Superstructure-General, for a discussion of structure types.

2. Span Arrangements. For stream crossings the desired minimum vertical clearance from high water to low chord is given in Chapter 8 - Hydraulics. Span lengths for multiple

span stream crossings are in most cases a matter of economics and the provision for an opening that adequately passes flood flows, ice and debris. For structures over waterways that qualify as navigable waters of the United States, the minimum vertical and horizontal clearances of the navigable span are determined by the U.S. Coast Guard after considering the interests of both highway and waterway transportation users.

For most of the ordinary grade separation structures the requirements for horizontal clearance determine the span arrangements. Refer to Chapter 17 - Superstructure-General for span length criteria.

3. Economics.

   Economy is a primary consideration in determining the type of structure to be used. Refer to Chapter 5 – Economics and Costs, for cost data.

   At some stream crossings where the grade line permits considerable head room, investigate the economy of a concrete box culvert versus a bridge type structure. When economy is not a factor, the box culvert is the preferred type from the standpoint of maintenance costs, highway safety, flexibility for roadway construction, and provision of a facility without roadway width restrictions.

4. Aesthetics. Recognition of aesthetics as an integral part of a structure is essential if bridges are to be designed in harmony with adjacent land use and development. Refer to Chapter 4 - Aesthetics.

5. Hydraulic Consideration. Stream crossing structures are influenced by stream flow, drift, scour, channel conditions, ice, navigation, and conservation requirements. This information is submitted as part of the Structure Survey Report. Refer to Chapter 8 - Hydraulics for Hydraulic considerations and Section 8.1.5 for Temporary Structure Criteria.

6. Geometrics of Design. The vertical and horizontal clearance roadway widths, design live loading, alignment, and other pertinent geometric requirements are given in Chapter 3.

7. Maintenance. All bridge types require structural maintenance during their service life. Maintenance of approaches, embankments, drainage, substructure, concrete deck, and minor facilities is the same for the various types of bridges. A minimum draining grade of 0.5% across the bridge is desirable to eliminate small ponds on the deck except for open railings where the cross slope is adequate.

   Epoxy coated bar steel is required in all new decks and slabs.

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

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

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

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

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

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

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

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

6.2.2.3 Requirements of Drawing

6.2.2.3.1 Plan View

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

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

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

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

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

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

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

7. Detail the extent of slope paving or riprap.

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

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

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

11. Median width if dual highway.

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

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

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

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

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

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

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

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

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

18. Structure numbers on plan.

19. Excavation protection for railroads.

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

21. Location of deck lighting or utilities if any.

22. Name Plate location.

23. Benchmark location

24. Locations of surface drains on approach pavement.

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

6.2.2.3.2 Elevation View

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

1. Profile of existing groundline or streambed.

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

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

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

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

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

7. Location and dimension of minimum vertical clearance.

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

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

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

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

10. Location of fixed and expansion bearings.

11. Location and type of expansion devices.

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

An elevation view is required for deck replacements, overlays with full-depth deck repair and painting plans (or any rehabilitation requiring the contractor to go beneath the bridge). Enough detail should be given to provide the contractor an understanding of what is beneath the bridge (e.g. roadway, bike path, stream, type of slope paving, etc.).

6.2.2.3.3 Cross-Section View

A view of a typical pier is shown as a part of the cross-section. The view shows the following general information:

1. Slab thickness, curb height and width, type of railing.
2. Horizontal dimensions tied into a reference line or centerline of roadway.
3. Girder spacing with girder depth.
4. Direction and amount of crown or superelevation, given in %.
5. Point referred to on profile grade.
6. Type of pier with size and number of columns proposed.
7. For solid, hammerhead or other type pier approximate size to scale.
8. Dimension minimum depth of bottom of footings below ditch or finished ground line or if railroad crossing below top of rail.
9. Location for public and private utilities to be carried in the superstructure. Label owner's name of utilities.
10. Location of lighting on the deck or under the deck if any.

6.2.2.3.4 Other Requirements

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

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

2. Channel change section if applicable. Approximate stream bed elevation at low point.
3. Any other view or detail which may influence the bridge type, length or clearance.
4. List design data including:

   <u>Material Properties</u>:

- 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: (if using LRFR)
Design Loading: HL-93
Inventory Rating Factor: RF = X.XX
Operating Rating Factor: RF = X.XX
Wisconsin Standard Permit Vehicle (Wis-SPV) = XXX kips

Live Load: (if using LFR)
Inventory Rating: HS = XX
Operating Rating: HS = XX
Wisconsin Standard Permit Vehicle (Wis-SPV) = XXX kips

(See Chapter 45 – Bridge Rating (45.9.3) for additional information)

Ratings (Plans Including Ratings that have not been changed)

Live Load: (if HSI uses LRFR)
Taken from HSI, xx/xx/2xxx
Design Loading: HL-93
Inventory Rating Factor: RF = X.XX
Operating Rating Factor: RF = X.XX
Wisconsin Standard Permit Vehicle (Wis-SPV) = XXX kips

Live Load: (if HSI uses LFR)
Taken from HSI, xx/xx/2xxx
Inventory Rating: HS = XX
Operating Rating: HS = XX

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

If widening a bridge, provide ratings for both the new and existing superstructure elements. For example, if widening a girder bridge previously designed with Load Factor Design, provide the LFR rating for the controlling existing girder and the LRFR rating for the controlling new girder.

Hydraulic Data

100 YEAR FREQUENCY

Q_{100} = XXXX C.F.S.
VEL. = X.X F.P.S.
HW_{100} = EL. XXX.XX
WATERWAY AREA = XXX SQ.FT.
DRAINAGE AREA = XX.X SQ.MI.
ROADWAY OVERTOPPING = (NA or add “Roadway Overtopping Frequency” data)
SCOUR CRITICAL CODE = X

2 YEAR FREQUENCY

Q_2 = XXXX C.F.S.
VEL. = X.X F.P.S.
HW_2 = EL. XXX.XX

ROAD OVERTOPPING FREQUENCY (if applicable, frequencies < 100 years)

FREQUENCY = XX YEARS
Q_{XX} = XXXX C.F.S.
HW_{XX}= EL. XXX.XX

(See Chapter 8 – Hydraulics for additional information)

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

6. Rehabilitation structure plans should use the same labeling convention as shown on the original structure plans when practical. Generally, this will include substructure labels (wings, abutments, piers, etc.) and girder numbers. This labeling convention is beneficial for inspection purposes.

6.2.2.4 Utilities

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

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

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

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

6.2.3 Distribution of Exhibits

6.2.3.1 Federal Highway Administration (FHWA).

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

1. Projects on the Interstate System

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

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

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

Unusual bridges have the following characteristics:

- Difficult or unique foundation problems

- New or complex designs with unique operational or design features

- Exceptionally long spans

- Design procedures that depart from currently recognized acceptable practices

Examples of unusual bridges:

- Cable-stayed

- Suspension

- Arch

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

Examples of unusual structures:

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

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

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

1. Preliminary plans (Type, Size and Location)
2. Bridge/structures related environmental concerns and suggested mitigation measures
3. Studies of bridge types and span arrangements
4. Approach bridge span layout plans and profile sheets
5. Controlling vertical and horizontal clearance requirements
6. Roadway geometry
7. Design specifications used
8. Special design criteria
9. Cost estimates

10. Hydraulic and scour design studies/reports showing scour predictions and related mitigation measures

11. Geotechnical studies/reports

12. Information on substructure and foundation types

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

6.2.3.2 Other Agencies

This is a list of other agencies that may or may not need to be coordinated with. There may be other stakeholders that require coordination. Consult FDM Chapter 5 for more details on coordination requirements.

- Department of Natural Resources

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

- Railroad (FDM Chapter 17)

  Begin communicating as early as possible with the Region Railroad Coordinator.

- Utilities (FDM Chapter 18, Bridge Manual Chapter 32)

  BOS discourages attachment of utilities to a structure. However, if there are no other viable options, private or public utilities desiring to attach their facilities (water, and sewer mains, ducts, cables, etc.) to the structure must apply to the owner for approval. For WisDOT owned structures, approval is required from the Region's Utilities & Access Management Unit.

- Coast Guard (FDM)

- Regions

  A copy of the preliminary plans is sent to the Regional Office involved for their review and use.

- Native American Tribal Governments

- Corps of Engineers

- Other governing municipalities

- State Historic Preservation Office
- Environmental Protection Agency
- Other DOTs

6.3 Final Plans

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

6.3.1 General Requirements

6.3.1.1 Drawing Size

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

6.3.1.2 Scale

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

6.3.1.3 Line Thickness

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

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

6.3.1.4 Lettering and Dimensions

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

6.3.1.5 Notes

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

6.3.1.6 Standard Insert Drawings

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

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

6.3.1.7 Abbreviations

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

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

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

Table 6.3-1
Abbreviations

6.3.1.8 Nomenclature and Definitions

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

6.3.2 Plan Sheets

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

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

1. General Plan
2. Subsurface Exploration
3. Abutments
4. Piers
5. Superstructure and Superstructure Details
6. Railing and Parapet Details

Show all views looking up station.

6.3.2.1 General Plan (Sheet 1)

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

1. Plan View

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

   a. Sufficient dimensions to layout structure in the field.

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

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

   On Structure Replacements

   Show existing structure in dashed-lines on Plan View.

2. Elevation View

   Same requirements as specified for preliminary plan except:

   a. Show elevation at bottom of all substructure units.

   b. Give estimated pile lengths where used.

3. Cross-Section View

   Same requirements as specified for preliminary plan except:

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

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

4. Grade Line

   Same requirements as specified for preliminary plan.

5. Design and Traffic Data

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

6. Hydraulic Information, if Applicable

7. Foundations

   Give soil/rock bearing capacity or pile capacity.

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

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

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

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

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

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

8. Estimated Quantities

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

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

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

9. General Notes

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

10. List of Drawings

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

11. 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.

12. Professional Seal

    All final bridge plans prepared by Consultants or Governmental Agencies shall be professionally sealed, signed, and dated on the general plan sheet. If the list of drawings is not on the general plan sheet, the sheet which has the list of drawings shall also be professionally sealed, signed, and dated. This is not required for WisDOT prepared plans, as they are covered elsewhere.

6.3.2.1.1 Plan Notes for New Bridge Construction

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

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

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

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

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

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

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

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

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

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

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

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

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

6.3.2.1.2 Plan Notes for Bridge Rehabilitation

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

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

9. 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. Wings should be numbered starting from the lower left corner and increasing in a clockwise order.

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.

Indicate girder numbers about the centerline of bearing in each span. Girders should be numbered in increasing order from left to right in the cross-section view. For rehabilitation projects, indicate the existing girder numbers and assign new girder numbers in increasing order from left to right.

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.

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

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

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

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

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

10. Provide a table of tangent offsets for the reference line and edges of deck at 10 foot intervals for curved bridges.

6.3.2.5.2 Steel Structures

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

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

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

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

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

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

6.3.2.5.3 Railing and Parapet Details

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

6.3.3 Miscellaneous Information

6.3.3.1 Bill of Bars

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

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

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

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

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

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

6.3.3.2 Box Culverts

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

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

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

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

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

6.3.3.3 Miscellaneous Structures

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

6.3.3.4 Standard Drawings

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

6.3.3.5 Insert Sheets

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

6.3.3.6 Change Orders and Maintenance Work

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

6.3.3.7 Name Plate and Benchmarks

For multi-directional bridges, locate the name plate on the roadway side of the first right wing or parapet traveling in the highway cardinal directions of North or East. For one-directional bridges, locate the name plate on the first right wing or parapet in the direction of travel. For type “NY”, “W”, “M” or timber railings, name plate to be located on wing. For parapets, name plate to be located on inside face of parapet.

A benchmark location shall be shown on bridge and larger culvert plans. Locate the benchmark on a horizontal surface flush with the concrete and in close proximity to the name plate. When possible, locate on top of the parapet on the bridge deck, above the abutment. Do not locate benchmarks at locations where elevations are subject to movement (e.g. midspan) and avoid placing below a rail or fence system. Benchmarks are typically metal survey disks, which are to be supplied by the department and set by the contractor. See FDM 9-25-5 for additional benchmark information.

6.3.3.8 Removing Old Structure and Debris Containment

This section provides guidance for selecting the appropriate Removing Old Structure bid item and determining when to use the "Debris Containment" bid item.

The "Removing Old Structure" bid item is most typically used for complete or substantial removals, as described in 6.3.3.8.2, of grade separation structures. In addition to this Standard Specification bid item, there are three STSP bid items for complete or substantial removal work over waterways: "Removing Old Structure Over Waterway", "Removing Old Structure Over Waterway With Minimal Debris", and "Removing Old Structure Over Waterway With Debris Capture System". The designer should review all of these STSPs and coordinate with the Wisconsin Department of Natural Recourses (DNR) to reach consensus on which STSP to use when removing a particular structure. **The designer should not automatically defer to the recommendation from the initial DNR letter, but should work with WisDOT and DNR environmental coordinators, considering constructability and cost impacts of the items.** For unique or difficult removals, designers should consult with the contracting community to assess costs and the feasibility of a particular removal technique. One of the following Removing Old Structure bid items should be selected for removals over waterways:

- Removing Old Structure Over Waterway is used where it is not possible to remove the structure without dropping it, or a portion of it, into a waterway or wetland; and that waterway or wetland is not highly environmentally sensitive. This special provision is typically appropriate for removing the following structure types: slab spans; voided slabs; cast-in-place concrete girder bridges; earth-filled bridges.

- Removing Old Structure Over Waterway With Minimal Debris is used where it is possible to remove the structure without dropping it, or a portion of it, into a waterway or wetland, and that waterway or wetland is not highly environmentally sensitive. This special provision is typically appropriate for removing all structures types except for the following bridges which are typically covered under Removing Old Structure Over Waterway: slab spans; voided slabs; cast-in-place concrete girder bridges; earth-filled bridges.

- Removing Old Structure Over Waterway With Debris Capture System is typically used when the waterway or wetland is highly environmentally sensitive. Before including this special provision in the contract, consult with the department's regional environmental coordinator to determine if the affected waterway or wetland is highly environmentally sensitive and if this special provision is appropriate.

Debris Containment is used where structure removal, reconstruction, or other construction operations may generate falling debris that might pose a safety hazard or environmental/contamination concern to facilities located under the structure. This item is most typically used where the removal area is located over a railroad.

The Debris Containment item is not used when one of the Removing Old Structure Over Waterway items is used.

6.3.3.8.1 Structure Repairs

Structure repair work could include, but is not limited to, the following bid items:

- Removing Concrete Masonry Deck Overlay
- Removing Asphaltic Concrete Deck Overlay
- Removing Polymer Overlay
- Cleaning Parapets
- Cleaning Concrete Surfaces
- Cleaning Decks to Reapply Concrete Masonry Overlay
- Preparation Decks (type)
- Cleaning Decks
- Joint Repair
- Curb Repair
- Concrete Surface Repair
- Full-Depth Deck Repair

Removal work limited to the above items is already included in the respective bid item specification, therefore a Removing Old Structure bid item not required. Use of Debris Containment should be reviewed for the following conditions:

- For work **over waterways**, a method of protecting the waterway is needed in some cases. Use Debris Containment, **only as needed** based on the extent and location of removal, and environmental sensitivity of the waterway. Debris is expected to be minimal.

- For work **over roadways**, Standard Specification, Sections 104 and 107, addresses safety of the traveling public and damage to all property, so generally no additional specifications are needed. **It is expected that pertinent lanes of the underpass roadway are closed when falling debris is expected from above.** No additional specifications are needed unless specifically requested with sufficient reason, in which case use Debris Containment **only as needed**, based on the extent and location of removal. Debris is expected to be minimal.

- For work **over railroads**, Standard Specification, Sections 104 and 107, addresses safety of the traveling public and damage to all property, so generally no additional specifications are needed. Exception: containment of debris is required where Full-Depth Deck Repair is expected. Use Debris Containment if Full-Depth Deck Repair is expected, or **only as needed**, based on the extent and location of removal. Debris is expected to be minimal.

6.3.3.8.2 Complete or Substantial Removals

Complete or substantial removals, not covered by one of the bid items listed in 6.3.3.8.1, should use a Removing Old Structure bid item. Substantial removals could include, but are not limited to; decks, parapets, and wingwalls. The appropriate Removing Old Structure bid item should be selected and the need for Debris Containment should be reviewed for the following conditions:

- For work **over waterways**, a method of protecting the waterway is needed if the removal area is located over the waterway. If the removal area is located over the waterway, use Removing Old Structure Over Waterway, Removing Old Structure Over Waterway With Minimal Debris or Removing Old Structure Over Waterway With Debris Capture System. If the removal area is not located over the waterway, use Removing Old Structure. The Debris Containment item is not used for this work.

- For work **over roadways**, Standard Specification, Sections 104 and 107, addresses safety of the traveling public and damage to all property, and Standard Specification, Section 203 Removing Old Culverts and Bridges addresses removal. **It is expected that pertinent lanes of the underpass roadway are closed when falling debris is expected from above.** Use Removing Old Structure. No additional specifications are needed unless specifically requested with sufficient reasoning. Use Debris Containment **only as needed**, based on the significance of the roadway and/or location of removal.

- For work **over railroads**, Standard Specification, Sections 104 and 107, addresses safety of the traveling public and damage to all property, and Standard Specification, Section 203 Removing Old Culverts and Bridges addresses removal. A method of protecting the railroad is needed if the removal area is located over the railroad. Use Removing Old Structure. Use Debris Containment if the **removal area is located over the railroad, or only as needed**, based on the extent and location of removal.

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 and computer runs
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
7. Final Hydrology and Hydraulic computations and structure sizing report
8. Contour map

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. Layout sheets
7. Elevation runs and bridge geometrics
8. Falsework plans*
9. Miscellaneous Test Report
10. 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. Computation of Quantities

6.4 Computation of Quantities

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

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 Granular Materials

Granular materials can be bid in units of tons or cubic yards. Structure plans should use the TON bid item for Structure Backfill, Granular Backfill, and Base Aggregate Dense 1 1/4-inch, unless directed otherwise by the Region. The Region may consider use of the CY bid item when contractor-provided tickets may be problematic or when the TON bid item is not used elsewhere on the contract. Other cases may also warrant the use of the CY bid item.

For Structure Backfill, Granular Backfill, and Base Aggregate Dense 1 1/4-inch materials use a 2.0 conversion factor (tons/cubic yard) for compacted TON bid items or use a 1.20 expansion factor (i.e. add an additional 20%) for CY bid items, unless directed otherwise. Refer to the FDM when preparing computations using other granular materials (breaker run, riprap, etc.).

Granular quantities and units should be coordinated with the roadway designer. For some structures, backfill quantities may be negligible to the roadway, while others may encompass a large portion of the roadway cross section and be present in multiple cross sections. A long

MSE retaining wall would be an example of the latter case and will require coordination with the roadway designer.

Generally, granular material pay limits should be shown on all structure plans. This information should be used to generate the estimated quantities and used to coordinate with roadway cross sections and construction details. See Standard Detail 9.01 – Structure Backfill Limits and Notes - for typical pay limits and plan notes.

Refer to 9.10 for additional information about granular materials.

6.4.3 Concrete Masonry Bridges

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

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

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

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

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

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

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

6.4.6 Bar Steel Reinforcement HS Stainless Bridges

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

6.4.7 Structural Steel Carbon or Structural Steel HS

See 24.2.4.

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

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

6.4.9 Piling Test Treated Timber (Structure)

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

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

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

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

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

6.4.11 Preboring CIP Concrete Piling or Steel Piling

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

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

Record the type and quantity, bid in lineal feet. For bridges, the railing length should be horizontal length shown on the plans. For retaining walls, use the length along the top of the wall. Calculate railing lengths as follows:

- Steel Railing Type ‘W’ – CL end post to CL end post
- Tubular Railing Type ‘H’ – CL end plate to CL end plate
- Combination Railing Type ‘3T’ – CL end post to CL end post + (2’-5”) per railing
- Tubular Railing Type ‘M’ – CL end post to CL end post + (4’-6”) per railing
- Combination Railing Type ‘Type C1-C6’ – CL end rail base plate to CL end rail base plate
- Tubular Steel Railing Type NY3&4 – CL end post to CL end post + (4’-10”) per railing

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

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

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

Record this quantity to the nearest 1 cubic yard.

6.4.15 Pile Points

When recommended in soils report. Bid as each.

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

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

6.4.17 Cofferdams (Structure)

Lump Sum

6.4.18 Rubberized Membrane Waterproofing

Record the quantity to the nearest square yard.

6.4.19 Expansion Devices

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

6.4.20 Electrical Work

Refer to Standard Construction Specifications for bid items.

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

Record this quantity in feet.

6.4.22 Preparation Decks Type 1 or Preparation Decks Type 2

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

6.4.23 Cleaning Decks

Record this quantity to the nearest square yard.

6.4.24 Joint Repair

Record this quantity to the nearest square yard.

6.4.25 Concrete Surface Repair

Record this quantity to the nearest square foot.

6.4.26 Full-Depth Deck Repair

Record this quantity to the nearest square yard.

6.4.27 Concrete Masonry Overlay Decks

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

6.4.28 Removing Old Structure and Debris Containment

For work over roadways and railroads, "Removing Old Structure" is most typically used for complete or substantial removals. For work over waterways, one of the following bid items should be used for complete or substantial removals: Removing Old Structure Over Waterway, Removing Old Structure Over Waterway With Minimal Debris, or Removing Old Structure Over Waterway With Debris Capture System. For work other than complete or substantial removals, a Removing Old Structure bid item may not be required.

Use Debris Containment, **only as needed** based on the significance, extent, or location of the removal.

See 6.3.3.8 for additional information on Removing Old Structure and Debris Containment bid items.

Bid as Lump Sum.

6.4.29 Anchor Assemblies for Steel Plate Beam Guard

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

6.4.30 Steel Diaphragms (Structure)

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

6.4.31 Welded Stud Shear Connectors X -Inch

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

6.4.32 Concrete Masonry Seal

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

6.4.33 Geotextile Fabric Type

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

6.4.34 Concrete Adhesive Anchors

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

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

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

6.4.36 Piling Steel Sheet Temporary

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

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

6.4.37 Temporary Shoring

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

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

6.4.38 Concrete Masonry Deck Repair

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

6.4.39 Sawing Pavement Deck Preparation Areas

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

6.4.40 Removing Bearings

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

6.4.41 Ice Hot Weather Concreting

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

6.4.42 Asphaltic Overlays

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

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

Coordinate asphaltic quantity assumptions with the Region and roadway designers.

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

On Federal (FHWA) or State Aid Projects (including maintenance projects), a completed Structure Survey Reports, preliminary and final plans are submitted to the Bureau of Structures with a copy forwarded to the Regional Office for review and approval prior to construction. Structure and project numbers are provided 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 loads on the structure.
- A means for verifying the completeness, constructability and accuracy of the structure plans.
- Verification that independent checks, reviews and ratings were performed.

The QA/QC plan shall also include the following items:

- Identification of a lead QA/QC Structures Program contact
- Identification of the QA/QC plan and procedures implementation date
- A statement indicating that the independent design check will be performed by an individual other than the designer, and the independent plan check will be performed by an individual other than the drafter.

Provisions for periodic reviews and update of the QA/QC plan with a frequency no less than 5 years; or as needed due to changes in the firm's personnel or firm's processes or procedures; or as requested by BOS.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.

Consultants' QA/QC plans and verification summary sheets may be subject to periodic reviews by BOS. These reviews are intended to assess compliance with BOS requirements listed above.

The list of consultant firms eligible to provide structural design services to WisDOT may be accessed using the link below:

https://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/strct/plan-submittal.aspx

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 Subsurface Investigation Report.
Consultant	Submit hydrology report via Esubmit or as an email attachment to the supervisor of the Consultant Review and Hydraulics Unit. Submit 60 days prior to preliminary plan submittal.
	Prepare preliminary plans according to 6.2.
	Coordinate with Region and other agencies per 6.2.3.
	Submit preliminary plans, SSR and supporting documents via e-submit for review and approval of type, size and location.
Structures Design Section	Record project information in HSIS.
	Review hydraulics for Stream Crossings.
	Review Preliminary Plan. A minimum of 30 days to review preliminary plans should be expected.
	Coordinate with other agencies per 6.2.3.
	Return preliminary plans and comments from Structures Design Section and other appropriate agencies to Consultant with a copy to the Regional Office.
	Forward Preliminary Plan and Hydraulic Data to DNR.
Consultant	Modify preliminary plan as required, and provide explanation for preliminary comments not incorporated in final plan.
	Prepare and complete final design and plans for the specified structure.
	Write special provisions.
	At least **two months** in advance of the PS&E date, submit the required final design documents via e-submit per 6.5.3.

Structures Design Section	Determine which final plans will be reviewed and perform quality assurance review as applicable.
	For final plans that are reviewed, return comments to Consultant and send copy to Regional Office, including FHWA as appropriate.
Consultant	Modify final plans and specifications as required.
	Submit modified final plans via e-submit as required.
Structures Design Section	Review modified final plans as applicable.
	Sign final plans and send performance evaluation form to Region and Consultant.
Geotechnical Consultant	At time of PSE, transmit gINT boring logs, soils laboratory testing summary and data sheets, and Soil Reports to the emails provided in the Soils and Subsurface Investigations section of Two/Three Party Design Contract Special Provisions.
Bureau of Project Development	Prepare final accepted structure plans for pre-development contract administration.
Consultant	If a plan change is needed after being advertised but before being let, an addendum is required per FDM 19-22-1 and 19-22 Attachment 1.2.
Structures Design Section	Review structure addendum as applicable.
	Sign structure addendum.
Bureau of Project Development	Distributes structure addendum to bidders.
Consultant	If a plan change is required after being let, a post-let revision is required per 6.5.5.
Structures Design Section	Review post-let revision as applicable.
	Stamp post-let revision plan as accepted.
	Delivers revised plan to DOT construction team for distribution.

Table 6.5-1
Approvals, Distribution and Work Flow

6.5.2 Preliminary Plan Requirements

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

1. Hydrology Report

2. Structure Survey Report

3. Preliminary plan, including log borings shown on the subsurface exploration sheet

4. Evaluation of subsurface investigation report

5. Contour map

6. Plan and profile, and typical section for roadway approaches

7. Hydraulic/Sizing Report (see Chapter 8 - Hydraulics) and hydraulic files are required for stream crossing structures

8. County map showing location of new and/or existing structures and FEMA map

9. Any other information or drawings which may influence location, layout or design of structure, including DNR initial review letter and photographs

6.5.3 Final Plan Requirements

The guidelines and requirements for Final Plan preparation are given in 6.3. The Load Rating Summary form and On-Time Submittal form can be found on the Bureau of Structures' Design and Construction webpage. The following files are included as part of the final-plan submittal:

1. Final Drawings

2. Design and Quantity Computations

   For all structures for which a finite element model was developed, include the model computer input file(s).

3. Special Provisions covering unique items not in the Standard Specifications or Standardized Special Provisions (STSP).

4. QA/QC Verification Sheet

5. Inventory Data Sheet

6. Bridge Load Rating Summary Form

7. LRFD Input File (Excel ratings spreadsheet)

8. On-Time Improvement Form

   The On-Time Improvement form is required to be submitted if either of the following situations occur:

   - If the first version of a final structure plan is submitted after the deadline of two months prior to the PSE date.

- If any version of a final structure plan is re-submitted after the deadline of two months prior to the PSE date. However this form is not required when the re-submit is prompted by comments from the Consultant Review Unit. The form is also not necessary when submitting addenda or post-let revisions.

6.5.4 Addenda

Addenda are plan and special provision changes that occur after the bid package has been advertised to potential bidders. See FDM 19-22-1 for instruction on the addenda process.

6.5.5 Post-Let Revisions

Post-let revisions are changes to plan details after the contract is awarded to a bidder. ESubmit only the changed plan sheets, not the entire plan set. The changes to the plan sheet shall be in red font, and outlined by red clouding. The revision box shall also be filled in with red font. Each sheet shall be 11x17, PE stamped, signed, and dated on the date of submittal.

6.5.6 Local-Let Projects

Local-let projects that are receiving State or Federal funding shall be submitted to and reviewed by the Consultant Review Unit in the same way as a State-let project. Final structure plans accepted and signed by the Consultant Review Unit will be returned to the Designer of Record and to the Region for incorporation into the local contract package.

6.6 Structures Data Management and Resources

6.6.1 Structures Data Management

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.

1. Structure Survey Report - Report is submitted by Region or Consultant and placed in the individual structure folder in HSI by BOS support staff.

2. Subsurface Exploration Report - Report is submitted by WisDOT Geotechnical Engineering Unit and placed in the individual structure folder in HSI by BOS support staff.

3. Hydraulic and Scour Computations, Contour Maps and Site Report - Data is assembled by the BOS Consultant Review & Hydraulics Unit and placed in the individual structure folder in HSI by BOS support staff.

4. Designer Computations and Inventory Superstructure Design Run (Substructure computer runs as determined by the Engineer). The designers record design, inventory, operating ratings and maximum vehicle weights on the plans.

5. Load Rating Input File and Load Rating Summary sheet - The designer submits an electronic copy of the input data for load rating the structure to the Structures Development Section. (For internal use, it is located at //H32751/rating.)

6. Structure Inventory Form (Available under "Inventory & Rating Forms" on the HSI page of the BOS website). New structure or rehabilitation structure 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.

7. 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 email to "DOTDTSDStructuresPiling@dot.wi.gov ". These two documents will be placed in HSI for each structure and can be found in the "Shop" folder.

8. Final Shop Drawings for steel bridges (highway and pedestrian), sign bridges, floor drains, railings, all steel joints, all bearings, high-mast poles, prestressed girders, prestressed boxes, noise walls and retaining walls. Metals Fabrication & Inspection Unit or others submit via email to the Structures Development Section at "DOTDLDTSDSTRUCTURESRECORDS@DOT.WI.GOV ". This process does not, however, supersede submission processes in place for specific projects.

9. Mill Tests, Heat Numbers and Shop Inspection Reports for all Steel Main Members. Metals Fabrication & Inspection Unit electronically files data into HSI

10. As-Let Plans: After bid letting, a digital image of the As-Let plans are placed in a computer folder in the Bureau of Project Development (BPD). BOS office support staff

extract a digital copy of the As-Let structure plans and place it in the structure folder for viewing on HSI.

11. As-Built Plans: As-Built structure plans shall be prepared for all let structure projects, new or rehabilitation. The structures with prefix 'B', 'P', 'C', 'M', 'N', 'R' and 'S' shall have As-Built plans produced after construction. The As-Built shall be prepared in accordance with Section 1.65.14 of the Construction and Materials Manual (CMM). These plans are located on a network drive and be viewed in DOTView GIS. BOS staff will ensure that the proper BOS folder (\\dotstrc\04bridge) has a copy of these plans for viewing in HSI.

12. Inspection Reports - A certified bridge inspector enters the initial and subsequent inspection data into HSI.

Initial	Underwater (UW-Probe/Visual)
Routine Visual	Movable
Fracture Critical	Damage
In-Depth	Interim
Underwater (UW)-Dive	Posted
Underwater (UW)-Survey	Structure Inventory and Appraisal

Table 6.6-1
Various Inspection Reports

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

6.6.2 Resources

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

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

Bridge Manual
Highway Structures Information System (HSI)
Insert sheets
Standard details
Posted bridge map
Standard bridge CADD files
Structure survey reports and check lists
Structure costs
Structure Special Provisions

https://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/strct/manuals.aspx

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

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

http://bridges.transportation.org

https://www.arema.org/

This page intentionally left blank.

Table of Contents

8.1 Introduction

The methods of hydrologic and hydraulic analysis provided in this chapter give the designer information necessary for an analysis of a roadway drainage crossing. Experience and sound engineering judgment are not to be ignored and may, at times, differ from results obtained using methods in this chapter. Very careful weighing of experience, judgment, and procedure must be made to arrive at a solution to the problem. Research in the field of drainage continues throughout the country and may subsequently alter the procedures found in this chapter.

8.1.1 Objectives of Highway Drainage

The objective of highway drainage is to prevent the accumulation and retention of water on and/or around the highway by:

- Anticipating the amount and frequency of storm runoff.
- Determining natural points of concentration of discharge and other hydraulic controls.
- Removing detrimental amounts of surface and subsurface water.
- Providing the most efficient hydraulic design consistent with economy, the importance of the road, maintenance and legal obligations.

8.1.2 Basic Policy

In designing highway drainage, there are three major considerations; first, the safety of the traveling public, second, the design should be in accordance with sound engineering practices to economically protect and drain the highway, and third, in accordance with reasonable interpretation of the law, to protect private property from flooding, water soaking or other damage. In general, the hydraulic adequacy of structures is determined by the methods as outlined in this manual and performance records of structures in the same or similar locations.

8.1.3 Design Frequency

Federal and State governments have placed increasing emphasis on environmental protection over the last several years. Consequently the administrative rules established by regulatory agencies have made past practice of designing structures to accommodate flood frequencies of 25 and 50 years obsolete and unworkable. Thus, the design discharge for all bridges and box culverts covered under this chapter shall be the 100 year (Q_{100}) frequency flood. In floodplain management this is also referred to as the Regional or Base flood. Design frequency is determined from requirements in Federal Highway Administration (FHWA) directives and the co-operative agreement between Wisconsin Department of Transportation (DOT) and Wisconsin Department of Natural Resources (DNR). The following publications are suggested for guidance.

8.1.3.1 FHWA Directive

Title 23, Chapter 1, Sub Chapter G, Part 650, Subpart A of the FHWA – Federal-Aid Policy Guide, "*Location and Hydraulic Design of Encroachments on Flood Plains*", prescribes FHWA policy and procedures. Copies of this directive may be found on the FHWA website.

8.1.3.2 DNR-DOT Cooperative Agreement

The Wisconsin Department of Transportation and the Wisconsin Department of Natural Resources have signed a co-operative agreement to provide a reasonable and economical procedure for carrying out their respective duties in a manner that is in the total public interest. The provisions in this agreement establish the basic considerations for highway stream crossings. A copy of this agreement can be found in Facilities Development Manual (FDM) 20-5-15.

8.1.3.3 DOT Facilities Development Manual

Refer to FDM Chapter 10 – Erosion Control and Storm Water Quality, FDM Chapter 11 – Design, FDM Chapter 13 - Drainage, and FDM Chapter 20 - Environmental Documents, Reports and Permits.

8.1.4 Hydraulic Site Report

The "Stream Crossings Structure Survey Report" shall be submitted for all bridge and box culvert projects. When submitting preliminary structure plans for a stream crossing, a hydraulic site report shall also be included. A check list of the various discussion items that need to be provided in the hydraulic site report is included as 8.6 Appendix 8-A. Plan survey datum must conform to datum in use by local zoning authorities. In most cases elevations are referenced to the National Geodetic Vertical Datum (NGVD) of 1929, or to the North American Vertical Datum of 1988 (NAVD 88). The Hydraulic Site Report discusses and documents the hydrologic, hydraulic, site conditions, and all other pertinent factors that influence the type, size, and location of the proposed structure.

8.1.5 Hydraulic Design Criteria for Temporary Structures

The basic design criteria for temporary structures will to be the ability to pass a 5-year storm (Q5) with only 0.5 feet of backwater over existing conditions. This criteria is only a general guideline and site specific factors and engineering judgment may indicate that this criteria is inappropriate. Separate hydraulic design criteria should be used for the design of temporary construction causeways. Factors that should be considered in the design of temporary structures and approach embankments are:

- Effects on surrounding property and buildings
- Velocities that would cause excessive scour
- Damage or inconvenience due to failure of temporary structure

- DNR concerns
- Temporary roadway profile
- Structure depths will be 36” for short spans and 48” or more for longer spans.

If possible and practical, the temporary roadway profile should be designed and constructed in such a manner that infrequent flood events are not obstructed from overflowing the temporary profile and creating excessive backwaters upstream of the construction. The temporary roadway profile should provide adequate clearance for the temporary structure.

The roadway designer should indicate the need for a temporary structure on the Stream Crossing Structure Survey Report. Preliminary and Final plans should indicate the hydraulic parameters of the temporary structure. The required parameters are the 5-year flood discharge (Q5), the 5-year high-water elevation (HW5), and the flow area of the temporary structure required to pass the 5-year flood (Abr).

8.1.6 Erosion Control Parameters

In order to assist designers in determining the appropriate erosion control measures to be provided at Bridge construction site, preliminary and final plans should indicate the 2-year flood discharge (Q2), 2-year velocity, and the 2-year high-water elevation (HW2).

8.1.7 Bridge Rehabilitation and Hydraulic Studies

Generally no hydraulic study will be required in bridge rehabilitation projects that do not involve encroachment to the Base Floodplain. This includes entire super structure replacement provided that the substructure and berm configuration remain unchanged and the low cord elevation is not significantly lowered.

The designer should consider historical high-water elevations, Flood Insurance Studies and the potential of inundation when choosing the replacement superstructure type. The risk of damage to the structure as the result of Scour should also be considered.

8.2 Hydrologic Analysis

The first step in designing a hydraulic structure is to determine the design discharge for the waterway. The problem is particularly difficult for small watersheds, say under five square miles, because the smaller the area, the more sensitive it is to conditions which affect runoff and the less likely there are runoff records for the area.

Acceptable methods of determining the design discharge for the 100 year flood shall be based on the guidelines contained in the *State Administrative Code NR 116.07, Wisconsin's Floodplain Management Program*[1]. Generally, a minimum of two methods should be used in determining a design discharge.

The most frequently used methods for determining the design discharge for bridges and box culverts in the State of Wisconsin are discussed below.

8.2.1 Regional Regression Equations

The U. S. Geological Survey (USGS) in cooperation with the Wisconsin Department of Transportation prepared a report entitled *Flood Frequency Characteristics of Wisconsin Streams*[2] which considers the flood potentials for a site using regional regression equations based on flood data from gaging stations on Wisconsin's rivers and streams. The flood-frequency regression equations are correlated with three or more of seven parameters, namely, drainage area, main-channel slope, storage, forest cover, mean annual snowfall, precipitation intensity index, and soil permeability. These equations are applicable to all drainage areas in Wisconsin except for highly regulated streams, and highly urbanized areas of the state.

8.2.2 Watershed Comparison

The results obtained from the above regression equations should be compared to similar gaged watersheds listed in reference (2) above using the area transfer formulas and procedures detailed in that document. A good discussion and examples of the use of regression equations and basin comparison methods can be seen in the WisDOT Facilities Development Manual, Procedure 13-10-5. The flood frequency discharges listed in reference (2) are for flood records up to the year 2000. More years of data are available from the USGS for most of the gaged watersheds.

The flood frequency discharges for the gaged watersheds can be updated past water year 2000 by using the Log-Pearson Type III distribution method as described in *Bulletin #17B entitled Guidelines For Determining Flood Flow Frequency*[3] and the guidelines for weighting the station skew with the generalized skew in *NR116.07, Wisconsin's Floodplain Management Program*[1].

8.2.3 Flood Insurance and Floodplain Studies

The Federal Emergency Management Agency (FEMA) had contracted for detailed flood studies throughout Wisconsin. They were developed for floodplain management and flood insurance purposes. These Flood Insurance Studies (FIS) which are on file with Floodplain-

Shoreland Management Section of the Wisconsin Dept. of Natural Resources (DNR) contain discharge values for many sites. These studies, along with other various floodplain studies, may be obtained from the DNR's Floodplain Analysis Interactive Map by using the following link:

https://dnr.wi.gov/topic/floodplains/mapindex.html

8.2.4 Natural Resources Conservation Service

For small watersheds in urban and rural areas, the National Resources Conservation Service (NRCS) has developed procedures to calculate storm runoff volumes, peak rates of discharge, hydrographs and storage volumes. The procedure is documented in *Technical Release 55 Urban Hydrology for Small Watersheds*[4].

8.3 Hydraulic Design of Bridges

Bridge design for roadway stream crossings requires analysis of the hydraulic characteristics for both the "existing conditions" and the "proposed conditions" of the project site. A thorough hydraulic analysis is essential to providing a properly sized, safe and economical bridge design and assessing the relative impact that the proposed bridge has on the floodplain. The following subsections discuss design considerations and hydraulic design procedures for bridges. See 8.6 Appendix 8-A for a checklist of items that need to be considered and included in the Hydraulic/Sizing report for stream crossing structures.

8.3.1 Hydraulic Design Factors

Several hydraulic factors dictate the design of both the bridge and the approach roadway within the floodplain limits of the project site. The critical hydraulic factors for design consideration are:

8.3.1.1 Velocity

Velocity through the bridge opening is a major design factor. Velocity relates to the scour potential in the bridge opening and the development of scour areas adjacent to the bridge. Examination of the "existing conditions" model, existing site conditions, soil conditions, and flooding history will give good insight to acceptable design velocity. Generally, velocities through bridges of less than 10 feet per second are acceptable.

8.3.1.2 Roadway Overflow

The vertical alignment of the approach grade is a critical factor in the bridge design when roadway overflow is a design consideration. The two important design features of roadway overflow is overtopping velocity and overtopping frequency. See 8.3.2.6.2

8.3.1.3 Bridge Skew

When a roadway is at a skew angle to the stream or floodway, the bridge shall also be at a skew to the roadway with the abutments and piers parallel to the flow of the stream. The hydraulic section through the bridge shall be the skewed section normal to the flow of the stream. Generally, in the design of stream crossing, the skew of the structure should be varied in increments of 5 degrees where practical. Improper skew can greatly aggravate the magnitude of scour.

8.3.1.4 Backwater and High-water Elevation

Roadways and bridges are generally restrictions to the normal flow of floodwaters and increase the flood profile in most situations. The increase in the flood profile is referred to as the backwater and the resultant upstream water surface elevation is referred to as the High-Water Elevation (HW).

The high-water elevation or backwater calculations at the bridge are directly related to the bridge size and roadway alignment, which dictates all of the aforementioned hydraulic design factors. A significant design consideration when computing backwater is the potential for increasing flood damage for upstream property owners. The Cooperative Agreement between the Wis. Department of Natural Resources (DNR) and Wis. Department of Transportation (DOT) (see 8.1.3.2) defines the policy for high-water elevation design. That portion of the Cooperative Agreement relating to floodplain considerations is based on the Wisconsin Adm. Rule NR116, “Wisconsin Floodplain Management Program”. It is advisable to thoroughly study both documents as they can significantly influence the hydraulic design of the bridge.

One very subtle backwater criteria which is not addressed under the guidelines of the DNR-DOT Cooperative Agreement, is the backwater produced for flood events less than the 100 year frequency flood. Design consideration should be given to the more frequent flood events when there is potential for increasing the extent and frequency of flood damage upstream.

8.3.1.5 Freeboard

Freeboard is defined as the vertical distance between the low cord elevation of the bridge superstructure and the high-water elevation. A freeboard of 2.0 feet is the desirable minimum for all types of superstructures. However, economics, vertical and horizontal alignment, and the scope of the project may force a compromise to the 2 foot minimum freeboard. For these situations, close evaluation shall be made of the type and amount of debris and ice that would pass through the structure. Freeboard should be computed using the low chord elevation at the upstream face on the lower end of the bridge. The calculated 100-year high water elevation at a cross section that is approximately one bridge length upstream should be used to check freeboard.

It has become common practice that if debris and ice are a potential problem, or adequate freeboard cannot be provided, a concrete slab superstructure is preferred. A girder superstructure may be susceptible to damage when ice and/or debris is a significant problem. Girder structures are more susceptible to damage associated with buoyancy and lateral hydrostatic forces. In situations where the superstructure may be inundated during major flood events, it is recommended that the girders be anchored, tied or blocked so they cannot be pushed or lifted off the substructure units by hydraulic forces. In addition, air vents near the top of the girder webs can allow entrapped air to escape and thus may reduce buoyancy forces. The use of Precast Pretensioned Slab and Box Sections is allowed where desirable freeboard cannot be provided and conventional cast in place slabs cannot be employed. The following requirements should be met:

- Precast Pretensioned Slab and Box Sections may be in the water for the 100-year flood. The designer will be responsible for ensuring the stability of the structure for buoyant and lateral forces.

- If Precast Pretensioned Slab and Box Sections are in contact with water for flood events equal to or less than a 5-year event, the Precast Pretensioned Slab and Box Sections must be cast solid.

- If Precast Pretensioned Slab and Box Sections are in contact with water for flood events equal to or less than a 100-year event, the void in Precast Pretensioned Slab and Box Sections must be cast with a non-water absorbing material.

8.3.1.6 Scour

Investigation of the potential for scour at the bridge site is a design consideration for the bridge opening geometry and size, as well as pier and abutment design. Bridges shall be designed to withstand the effects of scour from a super-flood (a flood exceeding the 100-year flood) without failing; i.e., experiencing foundation movement of a magnitude that requires corrective action. See 8.3.2.7. Generally, scour associated with a 100-year event without significant reduction in foundation factor of safety will accomplish this objective. For situations where a combination of flow through a bridge and over the roadway exist, scour should also be evaluated for flow conditions at the onset of flow over topping when velocity through the bridge may be the greatest.

8.3.2 Design Procedures

8.3.2.1 Determine Design Discharge

See 8.2 for procedures.

8.3.2.2 Determine Hydraulic Stream Slope

The primary method of determining the hydraulic slope of a stream is surveying the water surface elevation through a reach of stream 1500 feet upstream to 1500 feet downstream of the site. Intermediate points through this reach should also be surveyed to detect any significant slope variation.

There are situations, particularly on flat stream profiles, where it is difficult to determine a realistic slope using survey data. This will occur at normal water surface elevation at the mouth of a stream, upstream of a dam, or other significant restriction in the stream. In this case a USGS 7-1/2” quadrangle map and existing flood studies of the stream can be investigated to determine a reasonable stream slope.

8.3.2.3 Select Floodplain Cross-Section(s)

Generally, a minimum of two floodplain valley cross-section(s) are required to perform the hydraulic analysis of a bridge. The sections shall be normal to the stream flow at flood stage and approximately one bridge length upstream and downstream of the structure. A detailed cross-section of one or both faces of the bridge will also be required. If the section is skewed to the flow, the horizontal stationing shall be adjusted using the cosine of the skew angle.

If the downstream boundary condition of the hydraulic model is using normal depth, then the most downstream cross-section in the model should be located far enough downstream from the bridge and should reflect the natural floodplain conditions.

Field survey cross-sections will be needed when a contour map is plotted using stereographic methods. A field survey section is needed for that portion below the normal water surface.

Cross-sections taken from contour maps are acceptable when the information is supplemented with field survey sections and data. Additional sections may be required to develop a proper hydraulic model for the site.

The hydraulic cross-sections should not include slack water portions of the flood plain or portions not contributing to the downstream movement of water.

Refer to FDM 9-55 for a discussion of Drainage Structure Surveys.

8.3.2.4 Assign "Manning n" Values to Section(s)

"Manning n" values are assigned to the cross-section sub-areas. Generally, the main channel will have different "manning n" values than the overbank areas. Values are chosen by on-site inspection, pictures taken at the section, and use of aerial photos defining the extent of each "n" value. There are several published sources on open channel hydraulics which contain tables for selecting appropriate "n" values. See 8.5 References (5) and (6).

8.3.2.5 Select Hydraulic Model Methodology

There are several public and private computer software programs available for modeling open channel hydraulics, bridge hydraulics, and culvert hydraulics. Three public domain computer software programs that are most prevalent and preferred in Wisconsin bridge design work are "HEC-RAS", "WSPRO" and "HY8".

The HEC-RAS program is currently the most widely used methodology for floodplain and bridge hydraulic modeling. HEC-RAS has more options and capabilities than WSPRO when modeling complex floodplains and requires a greater amount of expertise to apply. HEC-RAS should be used where existing HEC-2 data is available from a previous Flood Insurance Study. The WSPRO methodology is tailored specifically for bridge hydraulics with many appropriate default coefficients and analysis options. More information on these two programs is given below. "HY8" is a FHWA sponsored culvert analysis package based on the FHWA publication "Hydraulic Design of Highway culverts" (HDS-5), see 8.5 Reference (13).

1. HEC-RAS

   The hydrologic Engineering Center's River Analysis System (HEC-RAS) is the first of the U.S. Army Corps of Engineers "Next Generation" software packages. It is the successor to the HEC-2 program, which was originally developed by the Corps of Engineers in the early 1970's. HEC-RAS includes several data entry, graphing, and reporting capabilities. It is well suited for modeling water flowing through a system of open channels and computing water surface profiles to be used for floodplain management and evaluation of floodway encroachments. HEC-RAS can also be used for bridge and culvert design and analysis and channel modification studies.

   For a complete treatise on the methodology of the program, see 8.5 reference (7), (8) and (9). The HEC-RAS program and supporting documentation can be downloaded from the U.S. Army Corps of Engineers web site: http://www.hec.usace.army.mil/software/hec-ras/. A list of vendors for HEC-RAS is also available on this web site.

2. WSPRO

   "Water Surface Profiles (WSPRO)" is a computer program developed by the U.S. Geological Survey under contract with Federal Highway Administration. WSPRO was specifically oriented toward hydraulic design of highway bridges although it is equally suitable for water surface profile computations unrelated to highway and bridge design.

   The program uses bridge backwater computations based on analyses documented in the USGS publication entitled Measurement of Peak Discharge at Width Contractions by Indirect Methods, see 8.5 reference (10).

   For a complete treatise on the methodology of the program, see 8.5 reference (11) and (12). The WSPRO program and supporting documentation can be downloaded from the following FHWA web site, or can be obtained through "McTrans" or "PcTrans". See 8.7 Appendix 8-B.

   http://www.fhwa.dot.gov/engineering/hydraulics/software.cfm

3. HY8

   HY8 is a computer program that uses the FHWA culvert hydraulic approaches and protocols as documented in the publication "Hydraulic Design Series 5: Hydraulic Design of Highway Culverts" (HDS-5). See 8.5 reference (13). HY8 can perform hydraulic computations for circular, rectangular, elliptical, metal box, high and low profile arch, as well as user defined geometry culverts. FHWA recently released a new Windows based version of the HY-8 culvert program. The methodology used by HY8 is discussed in 8.4.2.4. This program can be downloaded from the FHWA web site: http://www.fhwa.dot.gov/engineering/hydraulics/software.cfm.

8.3.2.6 Develop Hydraulic Model

First, a hydraulic model shall be developed for the "existing conditions" at the bridge site. This shall become the basis for hydraulic design of "proposed conditions" for the project and allows for an assessment of the relative hydraulic changes associated with the proposed structure. Special attention should be given to historic high-water and flood history, evidence of scour (high velocity), roadway overtopping, existing high-water, and compatibility with existing Flood Insurance Study (FIS) profiles. When current information and/or estimates of site conditions or flows differ significantly from adopted regulatory information (FIS), it may be necessary to compute both "design" and "regulatory" existing and proposed conditions.

There are a number of encompassing features of a steady state (flow is constant) hydraulic model for a roadway stream crossing. They include the natural adjacent floodplain, subject structure, any supplemental structures, and the roadway. Accurate modeling and calculations need to account for all potential conveyance mechanisms. Generally, most modern step-backwater methodologies can incorporate all of the above elements in the evaluation of hydraulic characteristics of the project site.

The designer shall determine whether the proposed site is located in a FEMA Special Flood Hazard Area (Zone AE, A, etc). If so, a determination shall be made whether an effective hydraulic model (HEC-RAS, HEC-2, WSPRO, etc) exists for the waterway. If an effective model exists, it shall be used to evaluate the impact of the proposed stream crossing structure on mapped floodplain elevations. Areas mapped as Zone AE should always have an effective model. Effective models can be acquired from the DNR or the FEMA Engineering Library. Contact a DNR regional floodplain engineer with any questions related to existing effective models.

The designer should verify that the results of the existing hydraulic model match the flood profile listed in the corresponding Flood Insurance Study (FIS) report. This is called the 'duplicate effective' model. The duplicate effective model should then be updated to include geometry based on any recent project survey information. This is called the 'corrected effective' model and will serve as the existing condition for the bridge hydraulic analysis.

The Project Engineer shall ensure the appropriate local zoning authority is notified of the results of the hydraulic analysis.

Official bridge hydraulic models and supporting documentation are available for download from the Highway Structures Information System (HSIS).

8.3.2.6.1 Bridge Hydraulics

The three most common types of flow through bridges are free surface flow (low flow), free surface (unsubmerged) orifice flow and submerged orifice flow. The latter two are also referred to as pressure flow. All of the above flow conditions may also occur simultaneously with flow over the roadway.

There are situations in which steep stream slopes are encountered and the flow may be supercritical (Froude No. > 1). This is a situation in which theoretically no backwater is created. For critical and supercritical flow situations the profile calculation would proceed from upstream to downstream. If this situation is encountered, the accuracy of the hydraulic model may be suspect and it is questionable whether the bridge should impose any constrictions on the stream channel. Sufficient clearance should be provided to insure that the superstructure will not come in contact with the flow.

Generally, in Wisconsin, most natural stream flow is in a sub-critical (Froude No. < 1) regime. Therefore, the water surface profile calculation will proceed from downstream to upstream.

Sample bridge hydraulic problems using HEC-RAS can be found in the HEC-RAS Applications Guide[9].

8.3.2.6.2 Roadway Overflow

One potential element in developing a hydraulic model for a stream crossing is roadway overflow. It is sometimes necessary to compute flow over highway embankments in combination with flow through structure openings. Most automated methodologies will incorporate the division of flow through a structure and over the road in determination of the solution. The WSPRO methodology will conduct the "combined flow" solution and internally determine and adjust the coefficient of discharge for both the structure and roadway weir section. Other methodologies, such as HEC-RAS, rely on user defined coefficients for both the structure and roadway flow solutions. The discharge equation and coefficients for flow over a highway embankment are given in this section.

The geometry and flow pattern for a highway embankment are illustrated in Figure 8.3-4. Under free flow conditions critical depths occur near the crown line. The head (H) is referred to the elevation of the water above the crown, and the length (L), in direction of flow, is the distance between the points of the upstream and downstream embankment faces (edge of shoulder). The length (B) of the embankment has no influence on the discharge coefficient.

The weir discharge equation is:

$$Q = k_t \cdot C_f \cdot B \cdot H^{3/2}$$

Where:

Q	=	discharge
C_f	=	coefficient of discharge for free flow conditions
B	=	length of flow section along the road normal to the direction of flow
H	=	total head = $h + h_V$
k_t	=	submergence factor

The length of overflow section (B) will be a function of the roadway profile grade line and depth of over-topping (h). Coefficient (C_f) is obtained by computing h/L and using Figure 8.3-1 or Figure 8.3-2, for paved or gravel roads.

The degree of submergence of a highway embankment is defined by ration ht/H. The effect of submergence on the discharge coefficient (C_f) is expressed by the factor k_t as shown in Figure 8.3-3. The factor k_t is multiplied by the discharge coefficient (C_f) for free-flow conditions to obtain the discharge coefficient for submerged conditions. For roadway overflow conditions with high degree of submergence, HEC-RAS switches to energy based calculations of the upstream water surface. The default maximum submergence is 0.95, however that criterion may be modified by the user.

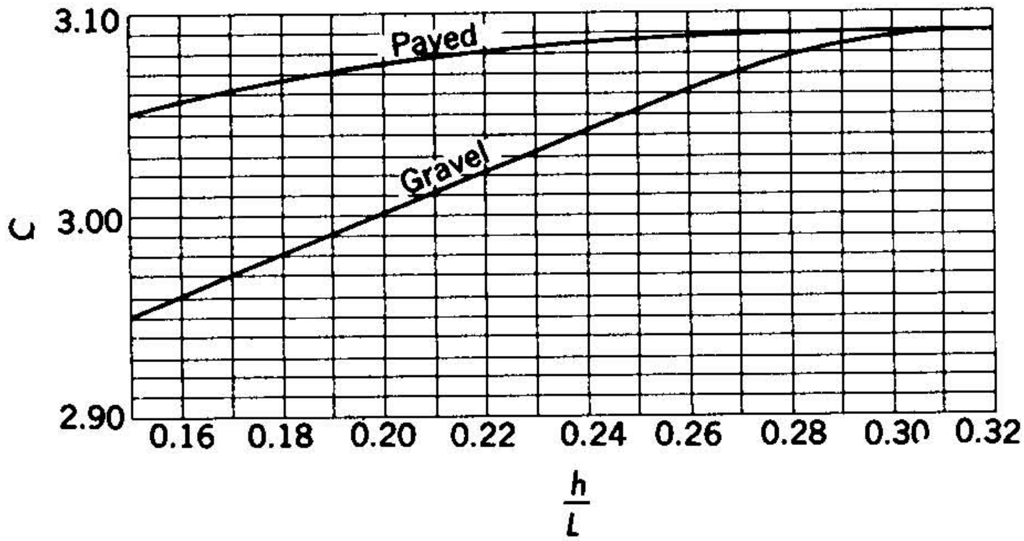


Figure 8.3-1

Discharge Coefficients, C_f, for Highway Embankments for H/L Ratios > 0.15

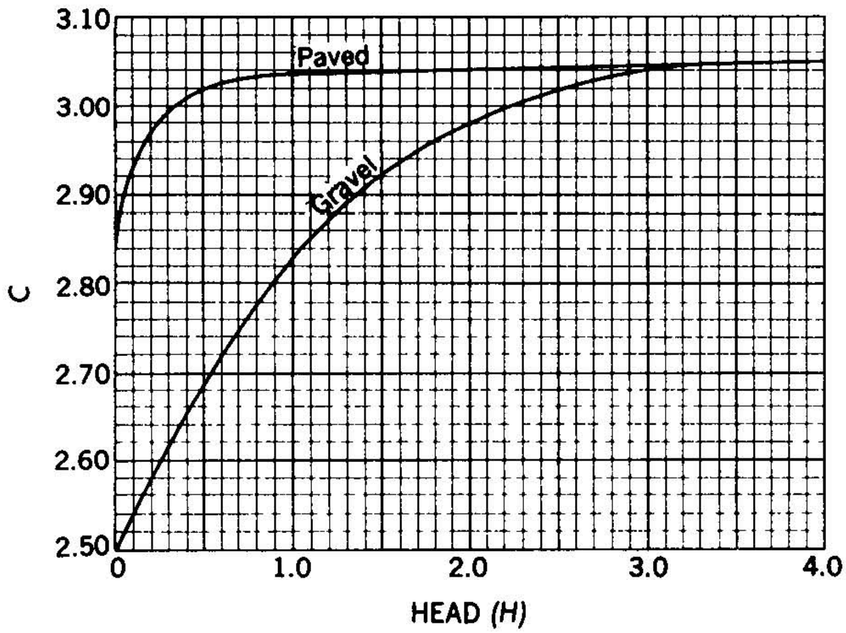


Figure 8.3-2
Discharge Coefficients, C_f, for Highway Embankments for H/L Ratios < 0.15

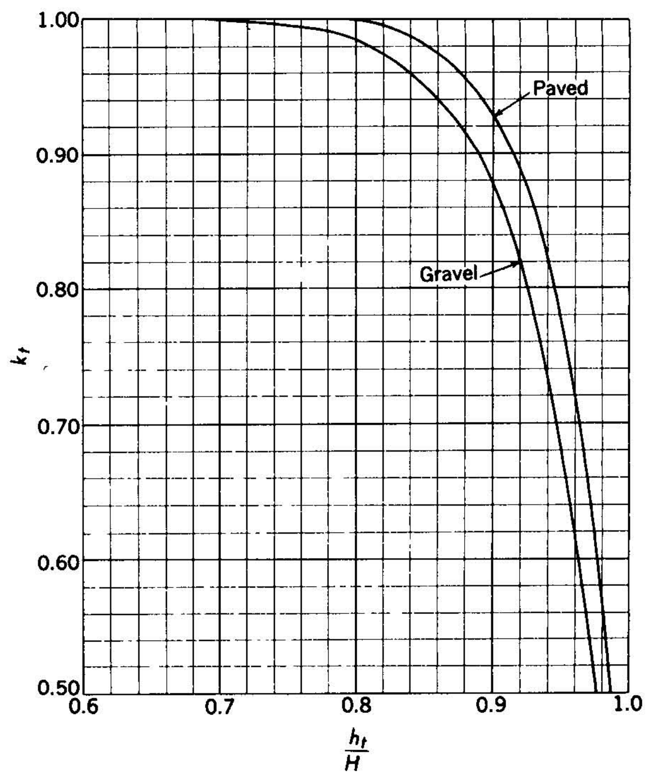


Figure 8.3-3
Definition of Adjustment Factor, k_t, for Submerged Highway Embankments

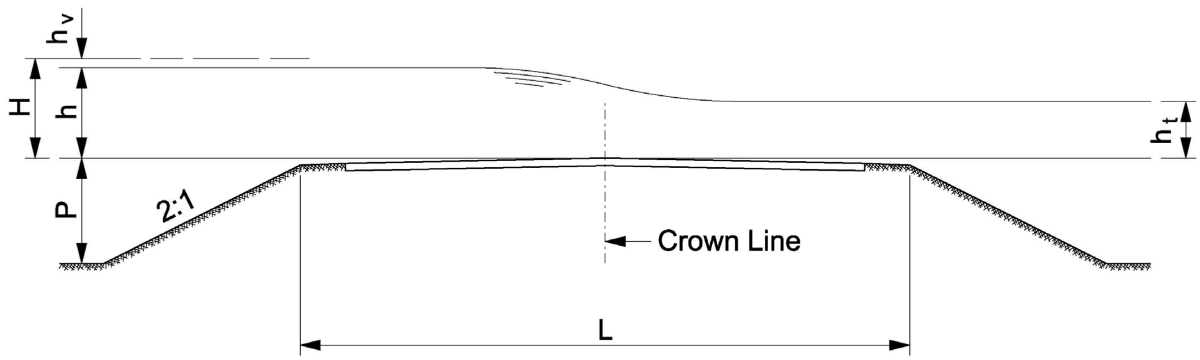


Figure 8.3-4
Definition Sketch of Flow Over Highway Embankment

8.3.2.7 Conduct Scour Evaluation

Evaluating scour potential at bridges is based on recommendations and background from FHWA Technical Advisory "*Evaluating Scour at Bridges*" dated October 28, 1991 and procedures from the *FHWA Hydraulic Engineering Circular No. 18, Evaluating Scour at Bridges, Fourth Edition*, May 2001[14], and *Hydraulic Engineering Circular No. 20, Stream Stability at Highway Structures, Third Edition*, March 2001[15]. Consult FHWA's website for the most current versions of the above publications.

All bridges shall be evaluated to determine the vulnerability to scour. In the FHWA publication *Recording and Coding Guide for Structure Inventory and Appraisal of the Nation's Bridges*[16], a code system has been established for evaluation. A section in this guide "Item 113 - Scour Critical Bridges" uses a single-digit code to identify the status of the bridge regarding its vulnerability to scour. The most current version of the Item 113 Scour Coding Guide can be found here: https://www.fhwa.dot.gov/engineering/hydraulics/policymemo/revguide.cfm.

There are three main components of total scour at a bridge site. They are Long-term Aggradation and Degradation, Contraction Scour, and Local Scour. In addition, lateral migration of the stream must be assessed when evaluating total scour at substructure units. Contraction and local scour will be evaluated in the context of clear-water and live bed scour conditions. In most of the methods for determining individual scour components, hydraulic characteristics at the approach section are required. The approach section should be understood as the cross section located approximately one bridge length upstream of the bridge opening.

8.3.2.7.1 Live Bed and Clear Water Scour

Clear-water scour occurs when there is insignificant or no movement (transport) of the bed material by the flow upstream of the crossing, but the acceleration of flow and vortices created by the piers or abutments causes the bed material in the vicinity of the crossing to move.

Live-bed scour occurs when there is significant transport of bed material from the upstream reach into the crossing.

8.3.2.7.2 Long-term Aggradation and Degradation

Aggradation is the deposition of eroded material in the stream from the upstream watershed. Degradation is the scouring (removal) of the streambed resulting from a deficient supply of sediment. These are subtle long term streambed elevation changes. These processes are natural in most cases. However, unnatural changes like dam construction or removal, as well as urbanization may cause Aggradation and Degradation. Excellent reference on this subject and the geomorphology of streams is the FHWA publication *Highways in the River Environment*[17], *HEC-18, Evaluating Scour at Bridges*[14], and *HEC-20, Stream Stability at Highway Structures*[15].

8.3.2.7.3 Contraction Scour

Generally, Contraction scour is caused by bridge approaches encroaching onto the floodplain and decreasing the flow area resulting in an increase in velocity through a bridge opening. The higher velocities are able to transport sediment out of the contracted area until an equilibrium is reached. Contraction scour can also be caused by short term changes in the downstream water surface elevation, such as bridges located on a meander bend or bridges located in the backwater of dams with highly fluctuating water levels. See 8.5 reference (14) & (15) for discussion and methods of analysis. If a pressure flow condition exists at the bridge opening, then vertical contraction scour must be evaluated. Reference HEC-18 for a description of the method used to estimate this scour component.

Computing Contraction Scour.

1. Live-Bed Contraction Scour

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{\frac{6}{7}} \left(\frac{W_1}{W_2}\right)^{k_1}$$

Where:

y_s	=	y_2-y_0 = Average scour depth, ft
y_1	=	Average depth in the upstream main Channel, ft
y_2	=	Average depth in the contracted section, ft

y_0 = Existing depth in the contracted section before scour, ft

Q_1 = Flow in upstream channel transporting sediment, ft^3/s

Q_2 = Flow in contracted channel, ft^3/s

W_1 = Bottom Width of upstream main channel, ft

W_2 = Net bottom Width of channel at contracted section, ft

k_1 = Exponent for mode of bed material transport, 0.59-0.69 see 8.5 ref. (14)

2. Clear-Water Contraction Scour

$$y_2 = \left[\frac{Q^2}{130 \cdot D_m^{\frac{3}{2}} \cdot W^2} \right]^{\frac{3}{7}}$$

Where:

y_s = y_2-y_0 = Average scour depth, ft

y_2 = Average depth in the contracted section, ft

y_0 = Existing depth in the contracted section before scouring, ft

Q = Discharge through the bridge associated with W, ft^3/s

D_m = Diameter of the smallest nontransportable particle ($1.25D_{50}$), ft

D_{50} = Median Diameter of the bed material (50% smaller than), ft

W = Net bottom Width of channel at contracted section, ft

8.3.2.7.4 Local Scour

Local scour is the removal of material from around a pier abutment, spur dike, or the embankment. It is caused by an acceleration of the flow and/or resulting vortices induced by obstructions to flow.

1. Pier Scour & Colorado State University's (CSU) Equation

   The recommended equation for determination of pier scour is the CSU's equation. Velocity is a factor in calculating the Froude Number. Therefore it is applicable where a hydraulic model of the bridge is available. The equation and appropriate charts and tables are shown below in Table 8.3-1, Table 8.3-2, Table 8.3-3 and Figure 8.3-5. See 8.5 reference (14) for a complete discussion of the CSU Equation.

The CSU equation for pier scour is:

$$\frac{y_s}{a} = 2.0 \cdot K_1 \cdot K_2 \cdot K_3 \cdot K_4 \cdot \left(\frac{y_1}{a}\right)^{0.35} \cdot Fr_1^{0.43}$$

Where:

y_s	=	Scour depth, ft
y_1	=	Flow depth directly upstream of the pier, ft
A	=	Pier width, ft
Fr_1	=	Froude number directly upstream of the pier = $V_1/(gy_1)^{1/2}$
V_1	=	Mean Velocity of flow directly upstream of the pier, ft/s
g	=	Acceleration of gravity, 32.2 ft/s^2
K_1	=	Correction Factor for pier nose shape (see Table 8.3-1 and Figure 8.3-5)
K_2	=	Correction Factor for angle of attack of flow (see Table 8.3-2)
K_3	=	Correction Factor for bed condition (see Table 8.3-3)
K_4	=	Correction Factor for armoring by bed material 0.7 - 1.0 (see 8.5 reference 14)

Correction Factor, K_1, for Pier Nose Shape (HEC-18 Table 2)	
Shape of Pier Nose	K_1
(a) Square Nose	1.1
(b) Round Nose	1.0
(c) Circular Cylinder	1.0
(d) Group of Cylinders	1.0
(e) Sharp Nose	0.9

Table 8.3-1
Correction Factor, K_1, for Pier Nose Shape

Correction Factor, K_2, for Angle of Attack, Θ, of the Flow (HEC-18 Table 3)			
Angle	L/a = 4	L/a = 8	L/a = 12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0
Angle = skew angle of flow L = length of pier, ft a = pier width, ft			

Table 8.3-2
Correction Factor, K_2, for Angle of Attack, θ, of the Flow

Increase in Equilibrium Pier Scour Depths, K_3, for Bed Conditions (HEC-18 Table 4)		
Bed Condition	Dune Height, ft	K_3
Clear – water Scour	N/A	1.1
Plane Bed and Antidune Flow	N/A	1.1
Small Dunes	3 > H ≥ 0.6	1.1
Medium Dunes	9 > H ≥ 3	1.2 to 1.1
Large Dunes	H ≥ 9	1.3

Table 8.3-3
Increase in Equilibrium Pier Scour Depths, K_3, for Bed Condition

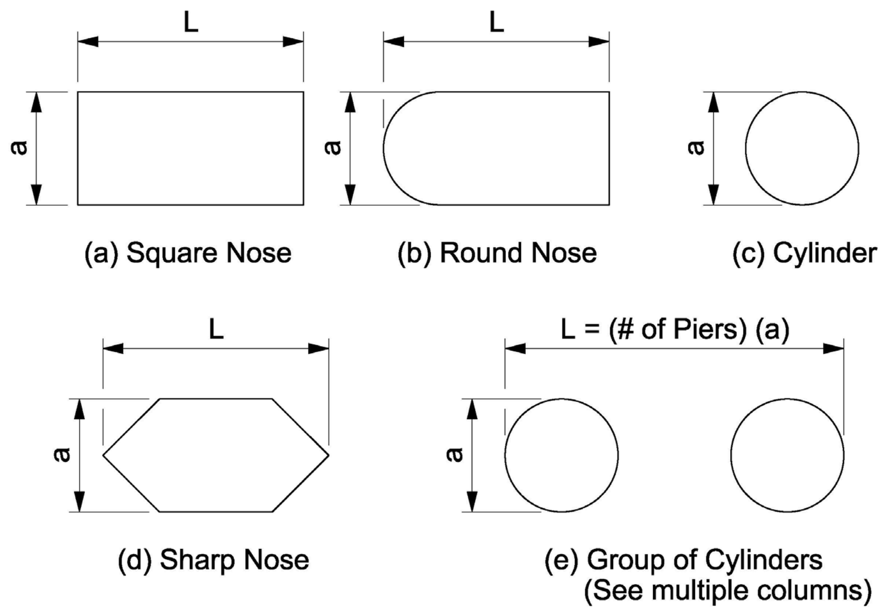


Figure 8.3-5
Common Pier Shapes

2. Abutment Scour Equations

Abutment scour analysis is dependent on equations that relate the degree of projection of encroachment (embankment) into the flood plain. Several equations were developed to estimate abutment scour depths, however lack of field data to verify any one equation causes doubt on the reliability of these scour estimates. This is one of the reasons heavy riprap underlain with geotextile fabric used to resist scour as described in the construction specifications at most stream crossing abutments.

Three methods are presented in the FHWA publication HEC-18 "Evaluating Scour at Bridges". The HIRE equation can be used when L/y1 is greater than 25, where L is the length of embankment projected and normal to the flow (ft), and y_1 is depth of flow at the abutment on the overbank or in the main channel (ft). For lower values of L/y1, the live-bed Froehlich equation can be used, which incorporates the effective embankment length. The user needs to refer to the publication HEC-18, see 8.5 reference (14), for a discussion of the applicability of the equations presented and further definitions of the parameters used in these equations. In addition, common hydraulic modeling programs used for bridge design, such HEC-RAS and WSPRO, include routines to calculate abutment scour. Designers are cautioned to closely examine how the parameters that are used in these automated routines are defined. The third approach presented in HEC-18 was recently developed under NCHRP Project 24-20. This method includes equations that encompass a range of abutment types and locations, as well as flow conditions. The primary advantage of this approach is that the equations are more physically representative of the abutment scour process, but it also avoids using the effective embankment length, which can be difficult to determine accurately. This approach computes total scour, rather than just local scour, at the abutment. Reference HEC-18 for a detailed description of the NCHRP approach and equations.

Froelich's Live-Bed Scour at Abutments

$$\frac{y_s}{y_a} = 2.27 \cdot K_1 \cdot K_2 \cdot \left(\frac{L'}{y_a}\right)^{0.43} \cdot F_r^{0.61} + 1$$

y_s = Scour depth, ft

y_a = Average depth of flow on the floodplain, ft

L' = Length of active flow obstructed by the embankment, ft

K_1 = Coefficient for abutment shape (see Table 8.3-4)

K_2 = Coefficient for angle of embankment to flow (see 8.5 reference 14)

Fr = Froude number of approach flow upstream of the abutment = $V_e/(gy_a)^{1/2}$

Where:

V_e = Q_e/A_e, ft/s

g = Acceleration of gravity, 32.2 ft/s^2

A_e = Flow Area of approach cross section obstructed by embankment, ft^2

Q_e = Flow obstructed by abutment and approach embankment, ft^3/s

The HIRE Equation for Live-Bed Scour at Abutments

$$\frac{y_s}{y_1} = 4 \cdot Fr_1^{0.33} \cdot \frac{K_1 \cdot K_2}{0.55}$$

y_s = Scour depth, ft

y_1 = Depth of flow at the abutment on the overbank or in the main channel, ft

K_1 = Coefficient for abutment shape (see Table 8.3-4)

K_2 = Coefficient for skew angle of abutment to flow (see 8.5 reference 14)

Fr_1 = Froude number based on velocity and depth adjacent to and upstream of the abutment

Description	K_1
Vertical-wall abutment	1.00
Vertical-wall abutment with wing walls	0.82
Spill-through abutment	0.55

Table 8.3-4
Abutment Shape Coefficients (K_1)

The Froehlich and HIRE equations often predict excessively conservative abutment scour depths. This is due to the fact that these equations were developed based on results of experiments in laboratory flumes and did not reflect the typical geometry or flow distribution associated with roadway encroachments on floodplains. However, since the NCHRP equations are more physically representative of the abutment scour process, greater confidence can be placed in the scour depths resulting from this approach.

8.3.2.7.5 Design Considerations for Scour

Provide adequate free board (2 feet desirable) to prevent occurrences of pressure flow conditions.

Pier foundation elevations on floodplains should be designed considering the potential of channel or thalweg migration over the design life of the structure.

Align all substructure units and especially piers with the direction of flow. Improper alignment may significantly increase the magnitude of scour.

Piers in the water should have a rounded or streamline nose to reduce turbulence and related scour potential.

Spill-through (sloping) abutments are less vulnerable to scour than vertical wall abutments.

The Froehlich and HIRE equations used to estimate the magnitude of abutment scour were developed in a laboratory under ideal conditions and lack adequate field verification. These equations may tend to over estimate the magnitude of scour. These equations should be incorporated with a great deal of discretion.

8.3.2.8 Select Bridge Design Alternatives

In most design situations, the “proposed bridge” design will be based on the various pertinent design factors discussed in 8.3.1. They will dictate the final selection of bridge length, abutment design, superstructure design and approach roadway design. The Hydraulic/Site report should adequately document the site characteristics, hydrologic and hydraulic calculations, as well as the bridge type and size alternatives considered. See 8.6 Appendix 8-A for a sample check list of items that need to be included in the Hydraulic/Site Report.

8.4 Hydraulic Design of Box Culverts

Box culverts are an efficient and economical design alternative for roadway stream crossings with design discharges in the 300 to 1500 cfs range. As a general guide culvert pipes are best suited for smaller discharge values while bridges are better suited for larger values. Although multi-cell box culverts are designed for larger discharges, the larger size culverts tend to lose the hydraulic and economic advantage over bridges. The following subsections discuss the design considerations and hydraulic design procedures for box culverts.

8.4.1 Hydraulic Design Factors

As in the hydraulic design of bridges, several hydraulic factors dictate the design of both the culvert and approach roadway. The critical hydraulic factors for design considerations are:

8.4.1.1 Economics

The best economics for box culvert design are realized with the culvert flowing full and producing a reasonable headwater depth (HW) within the boundary of other hydraulic and roadway design constraints.

For long box culverts, particularly on steep slopes, considerable savings can be realized by incorporating an improved inlet design known as “Tapered Inlets”. The improved efficiency of the inlet where the inlet controls the headwater, will allow for design of a smaller culvert barrel. See 8.5 reference (13) for discussion on “Tapered Inlets”.

8.4.1.2 Minimum Size

If the highway grade permits, a minimum five foot box culvert height is desirable for clean-out purposes.

8.4.1.3 Allowable Velocities and Outlet Scour

Generally, for velocities under 10 fps no riprap is needed at the discharge end of a box culvert, although close examination of local soil conditions is advisable.

For outlet velocities from 10-14 fps heavy riprap shall be used extending 15 to 35 feet from the end of the culvert apron.

For velocities greater than 14 fps energy dissipators should be considered. These are the most expensive means of end protection. See 8.4.2.7 for the hydraulic design of energy dissipators.

When heavy riprap is used it is carried up the slopes around the ends of the outlet apron to an elevation at mid-length of apron wing.

8.4.1.4 Roadway Overflow

See 8.3.1.2.

8.4.1.5 Culvert Skew

See 8.3.1.3.

8.4.1.6 Backwater and Highwater Elevations

The “Highwater elevation” commonly referred to as headwater for culverts, is the backwater created at the upstream end of the culvert. Although culverts are more hydraulically efficient and economical when flowing under a reasonable headwater, several factors shall be considered in determining an allowable highwater elevation. For further discussion see Section 8.3.1.4.

8.4.1.7 Debris Protection

Debris protection is provided where physical study of the drainage area indicates considerable debris collection. Where used, structural design of debris protection features should be part of the culvert design. The box culvert survey report must justify the need for protection. Sample debris protection devices are presented in the FHWA publication, *Hydraulic Engineering Circular No. 9, Debris Control Structures, Evaluation and Countermeasures.* See 8.5 reference (18).

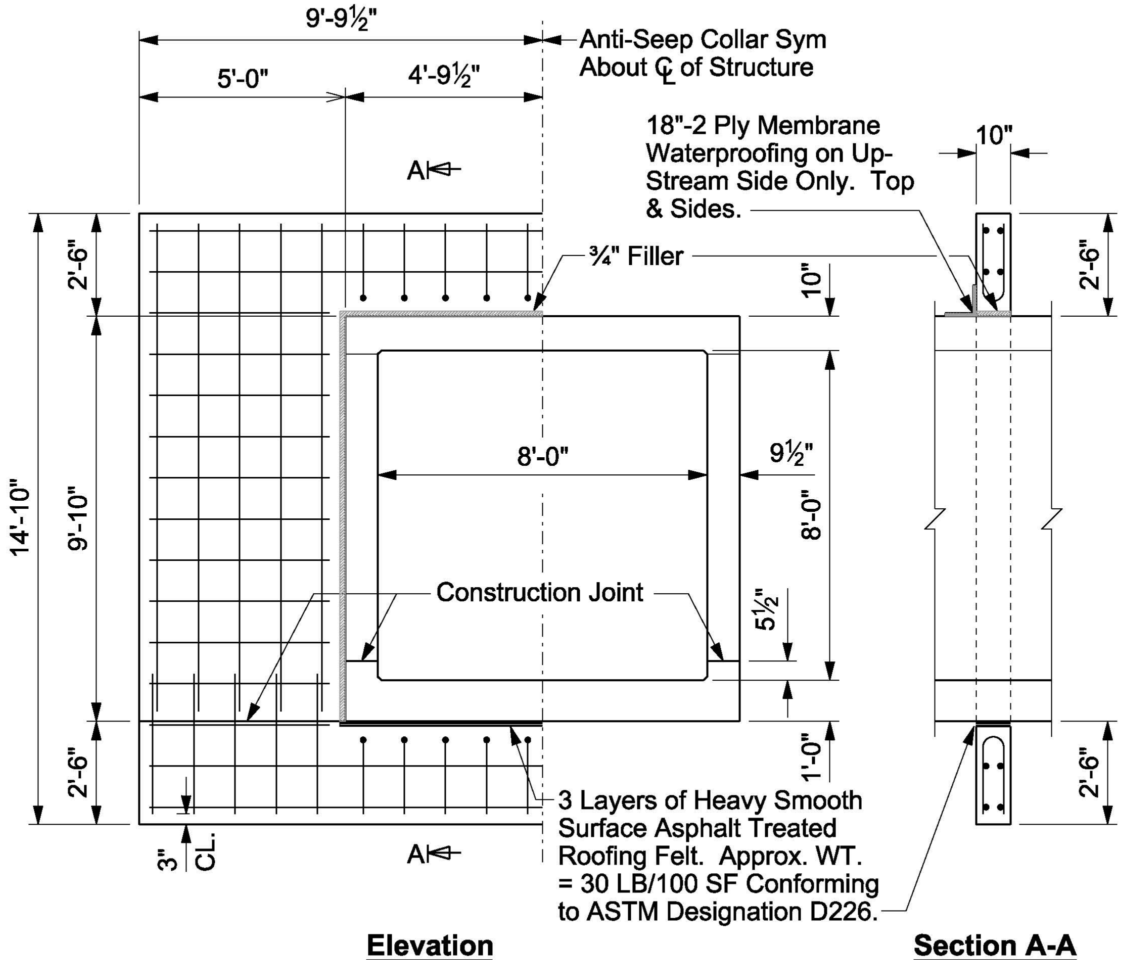
8.4.1.8 Anti-Seepage Collar

Anti-seepage collars are used to prevent the movement of water along the outside of the culvert and the failure by piping of the fill next to the culvert. They are used in sandy fills where the culvert has a large headwater.

Collars are located at the midpoint and upstream quarter point on long box culverts. If only one collar is used, it is located far enough from the inlet to intercept the phreatic (zero pressure) line to prevent seepage over the top of the collar. See 8.5 reference (19).

A typical collar is shown in Figure 8.4-1 and is applicable to all single and twin box structures.

An alternate method of preventing seepage is to use a minimum one foot thick impervious soil blanket around the culvert inlet extending five feet over undisturbed embankment. The same effect can be obtained by designing seepage protection into the endwalls.



All Bars Are #4s Spaced at 1'-0"

Figure 8.4-1
Anti-Seepage Collar

8.4.1.9 Weep Holes

The need for weep holes should be investigated for clay type soils with high fills, and should be eliminated in other cases.

If weep holes are necessary, alternate layers of fine and coarse aggregate are placed around the holes starting with coarse aggregate next to the hole.

8.4.2 Design Procedure

8.4.2.1 Determine Design Discharge

See 8.2 for procedures.

8.4.2.2 Determine Hydraulic Stream Slope

See 8.3.2.2 for procedures.

8.4.2.3 Determine Tailwater Elevation

The tailwater elevation is the depth of water in the natural channel computed at the outlet of the culvert. In situations of steeper slopes and small culverts, the tailwater is not a critical design factor. However, for mild slopes and larger culverts, the tailwater is a critical design factor. It may control the outlet velocity and depth of flow in the culvert.

The tailwater elevation is calculated using a typical section downstream of the outlet and performing a “normal depth” analysis. Most hydraulic engineering textbooks and handbooks include discussion of methods to calculate “normal depth” for symmetrical and irregular cross-sections in an open channel.

8.4.2.4 Design Methodology

The most prevalent design methodology for culverts is the procedure in the FHWA publication DHS No. 5, see 8.5 reference (13). It is highly recommended the designer first thoroughly study the methodologies presented in that publication.

Several computer software programs are available from public and private sources which use the same technique and methodology presented in HDS No. 5. One public domain computer program developed by FHWA entitled “HY8” is based on the HDS No. 5 manual. This program and documentation are available from the FHWA web site (see 8.7 Appendix 8-B). HEC-RAS and WSPRO also have culvert options using the same methodology. These programs have the capability of allowing the user to calculate the tailwater based on a downstream section and to calculate a combination of culvert and roadway overflow.

8.4.2.5 Develop Hydraulic Model

There are two major types of culvert flow: (1) flow with inlet control, and (2) flow with outlet control. For each type of control, different factors and formulas are used to compute the hydraulic capacity of a culvert. Under inlet control, the cross-sectional area, and the inlet geometry at the entrance are of primary importance. Outlet control involves the consideration of the tailwater in the outlet channel, the culvert slope, the culvert roughness, and the length of the culvert barrel, as well as inlet geometry and cross-sectional area.

Another design of Inlet control which is used frequently is “Tapered Inlets” or improved inlets. The slope-tapered and side-tapered inlets are more efficient hydraulically, and can be a more economical design for long culverts in flow with inlet control.

In all culvert design, headwater depth (HW) or depth of water at the entrance to a culvert is an important factor in culvert capacity. The headwater depth is the vertical height from the culvert invert elevation at the entrance to the total energy elevation of the headwater pool (depth plus velocity head). Because of the low velocities at the entrance in most cases and difficulty in determining the velocity head for all flows, the water surface elevation and the total energy elevation at the entrance are assumed to be coincident.

The box culvert charts presented here are inlet and outlet control nomographs Figure 8.4-3 and Figure 8.4-4, and a critical depth chart Figure 8.4-6. Note the “Inlet Type” over the HW/D scales on Figure 8.4-3 and entrance loss coefficients “Ke” for inlet types on Figure 8.4-4. The following illustrative problems are examples of their use. Forms similar to Figure 8.4-2 are used for computation.

1. Outlet Control Problem.

   The information necessary to solve this problem is given in Figure 8.4-2.

   Check for Inlet Control: For a Q/B value of 36 and a twin 10 x 5 box with type “C” inlet; HW/D=1.08 from Figure 8.4-3.

   The HW = 1.08 (5 ft) = 5.4 ft.

   Check for Outlet Control: For Q = 720/2 = 360 cfs. Length = 180 ft. and type “C” inlet; H = 1.5 ft. from Figure 8.4-4, TW = 5.2 ft. = ho

   Then HW = H + h_o - LS_o = 1.5 ft. + 5.2 ft. - .2 ft. = 6.5 ft.

   Design HW is 6.5 ft. (outlet controls) and the outlet velocity is 7.2 f.p.s. No heavy riprap is needed at the discharge apron.

2. Inlet Control Problem.

   The information necessary to solve this problem is given in Figure 8.4-5.

   Check for Inlet Control: For a Q/B value of 36 and a twin 10 x 5 box with type “C” inlet; HW/D = 1.08 from Figure 8.4-3.

   Then HW = 1.08 (5 ft.) = 5.4 ft.

   Check for Outlet Control: For Q = 720/2 = 360 cfs. Length = 132 ft. and type “C” inlet; H = 1.3 ft. from Figure 8.4-4. From Figure 8.4-6 critical depth = 3.4 ft. ho = (3.4 ft. + 5 ft.)/2 = 4.2 ft.

   Then HW = H + h_o - LS_o = 1.3 ft. + 4.2 ft. - .7 ft. = 4.8 ft.

   Design HW = 5.4 ft. (inlet control) and the outlet velocity is 11.0 f.p.s. Heavy riprap is needed at the discharge apron.

HYDROLOGIC AND CHANNEL INFORMATION

State of Wisconsin/Department of Transportation
E-B-31-68

Project: Outlet Control Problem
Culvert Sta.: 560+00

Designer: L. J. G.
Date: 9-23-69

Hydrology:
(50 freq.) Q = 720 cfs.
(___ freq.) Q = ___ cfs.

Elev. 876.87 ft.
875.5
AHW 6.5 ft
Elev. 869.00 ft.
Slope (S_0) .001 ft./ft.
Length 180 ft.
Elev. 868.82 ft.
Tailwater 5.2 ft.

874.0
868.8
5.2

$\frac{L}{100 S_0} = \frac{180}{.1} =$ ___

Location comments: The tailwater is controled by the inlet control problem which is downstream a short distance.

Culvert: Entrance	Culvert: Material	Culvert: Size	Q	Capacity Charts HW	Inlet Cont.: $\frac{HW}{D}$	Inlet Cont.: HW	Outlet Control: K_e	Outlet Control: d_c	Outlet Control: $\frac{d_c+D}{2}$	Outlet Control: h_o*	Outlet Control: H	Outlet Control: LS_o	Outlet Control: **HW	Controling HW	Outlet Velocity f.p.s.	Comments
Type "C"	R.C. Box	TWIN 10x5	36 720		1.08	5.4	0.2	3.4	4.2	5.2	1.5	.2	6.5	6.5	7.2	(For n=.015)

* h_o = The greater of $\frac{d_c+D}{2}$ or TW

** $HW = H + h_o - LS_o$

Summary & Recommendations:

Figure 8.4-2
Culvert Computation Form

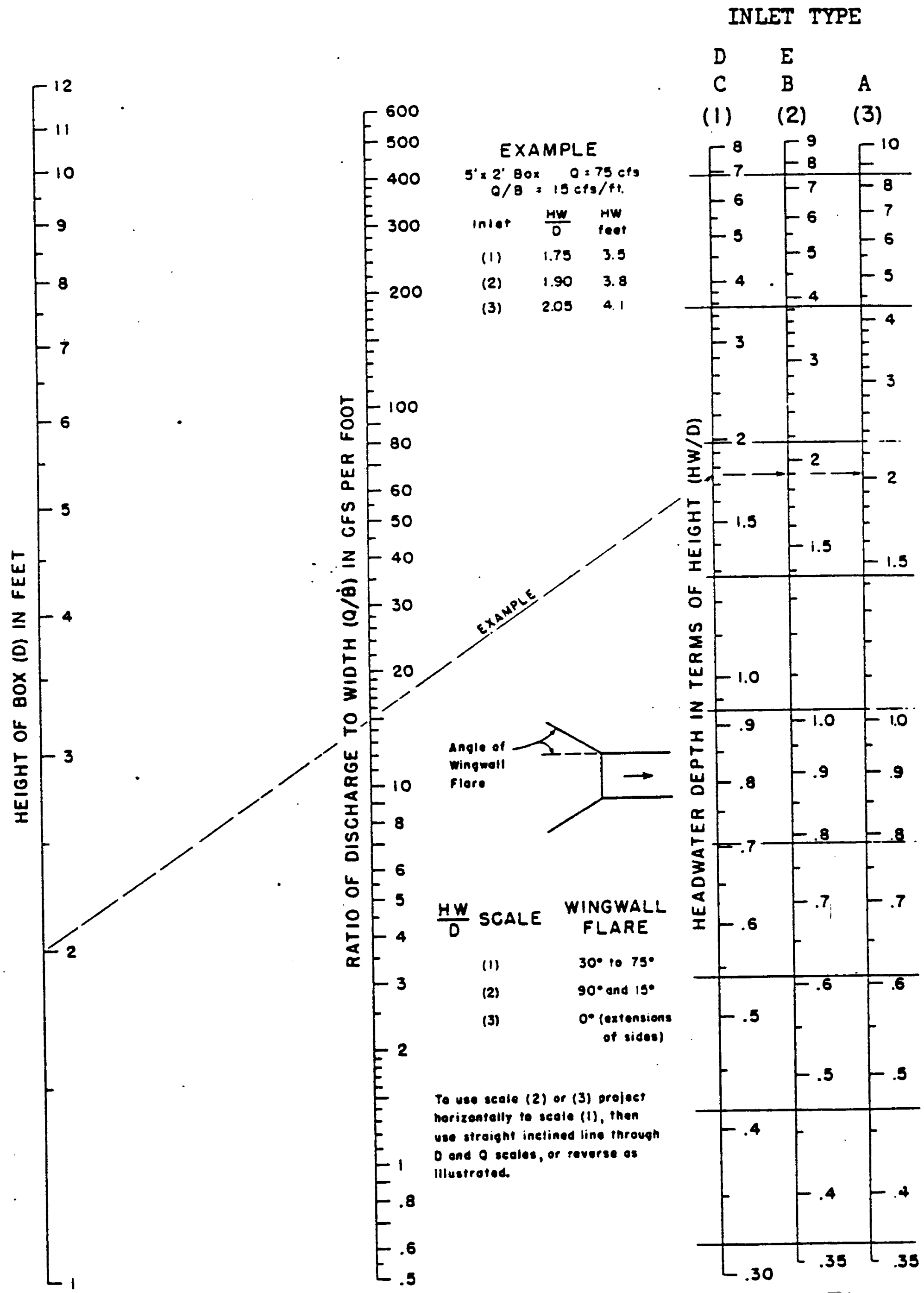


Figure 8.4-3
Headwater Depth for Box Culverts with Inlet Control

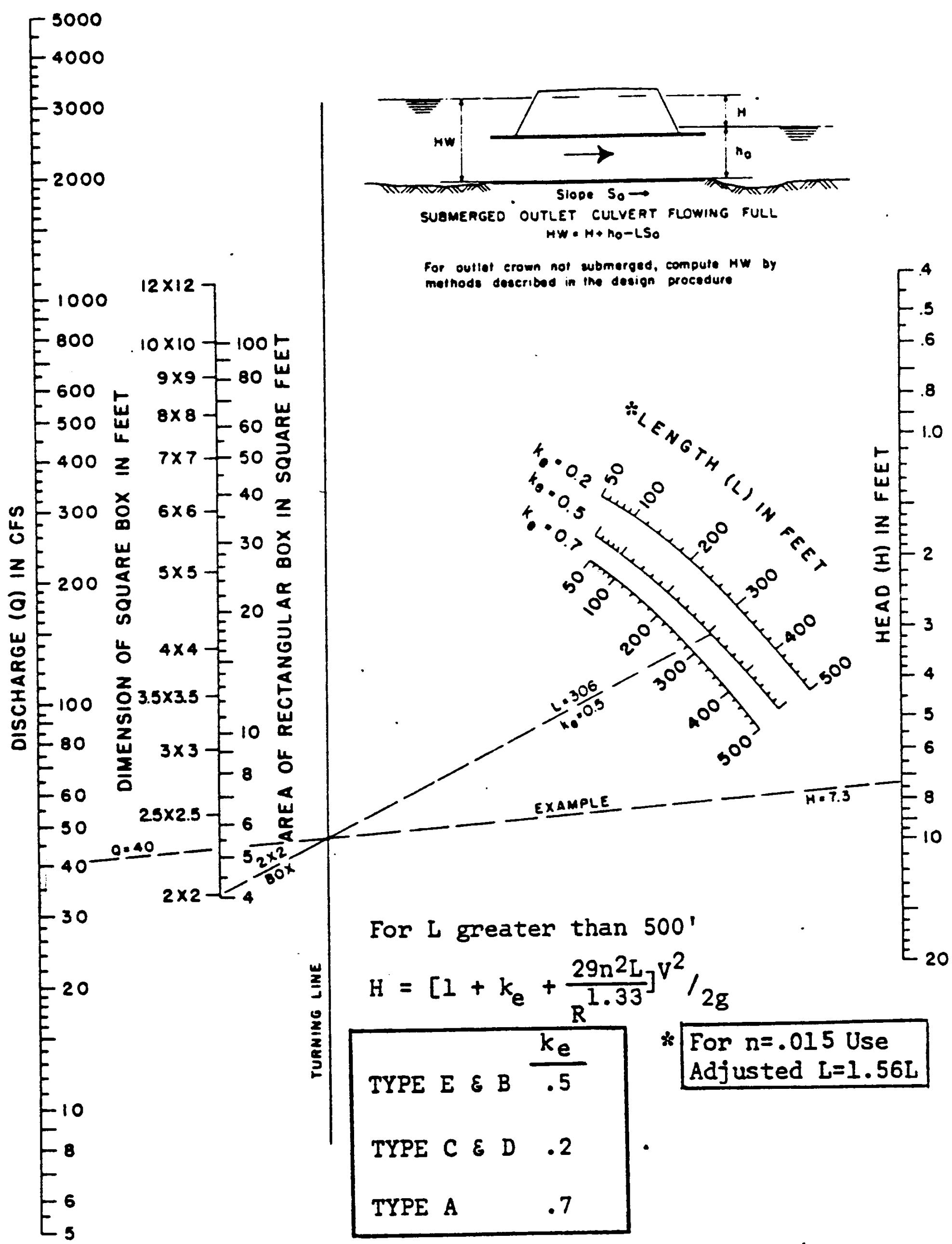


Figure 8.4-4

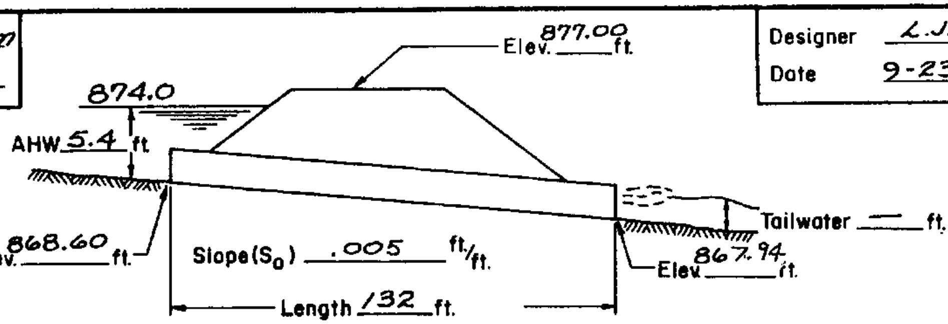
Head for Concrete Box Culverts Flowing Full, n = 0.012

HYDROLOGIC AND CHANNEL INFORMATION

State of Wisconsin/Department of Transportation
F-B-31-68

Project *Inlet Control Problem*
Culvert Sta. *432+00*

Designer *L.J.G.*
Date *9-23-69*



Hydrology:
(50 freq.) Q = 720 cfs.
(____ freq.) Q = ____ cfs.

$\frac{L}{100 S_o}$ = ________ = ________

Location comments: *This structure is located a short distance downstream of the outlet control problem.*

Culvert: Entrance	Culvert: Material	Culvert: Size	Q	Capacity Charts HW	Inlet Cont.: $\frac{HW}{D}$	Inlet Cont.: HW	Outlet Control: K_e	Outlet Control: d_c	Outlet Control: $\frac{d_c+D}{2}$	Outlet Control: h_o*	Outlet Control: H	Outlet Control: LS_o	Outlet Control: **HW	Controlling HW	Outlet Velocity f.p.s.	Comments
Type "C"	R.C. Box	TWIN 10x5	36 720		1.08	5.4	0.2	3.43	4.2	4.2	1.3	.7	4.8	5.4	11.0	(n= .015)

* h_o = The greater of $\frac{d_c+D}{2}$ or TW

** $HW = H + h_o - LS_o$

Summary & Recommendations:

Figure 8.4-5
Culvert Computation Form

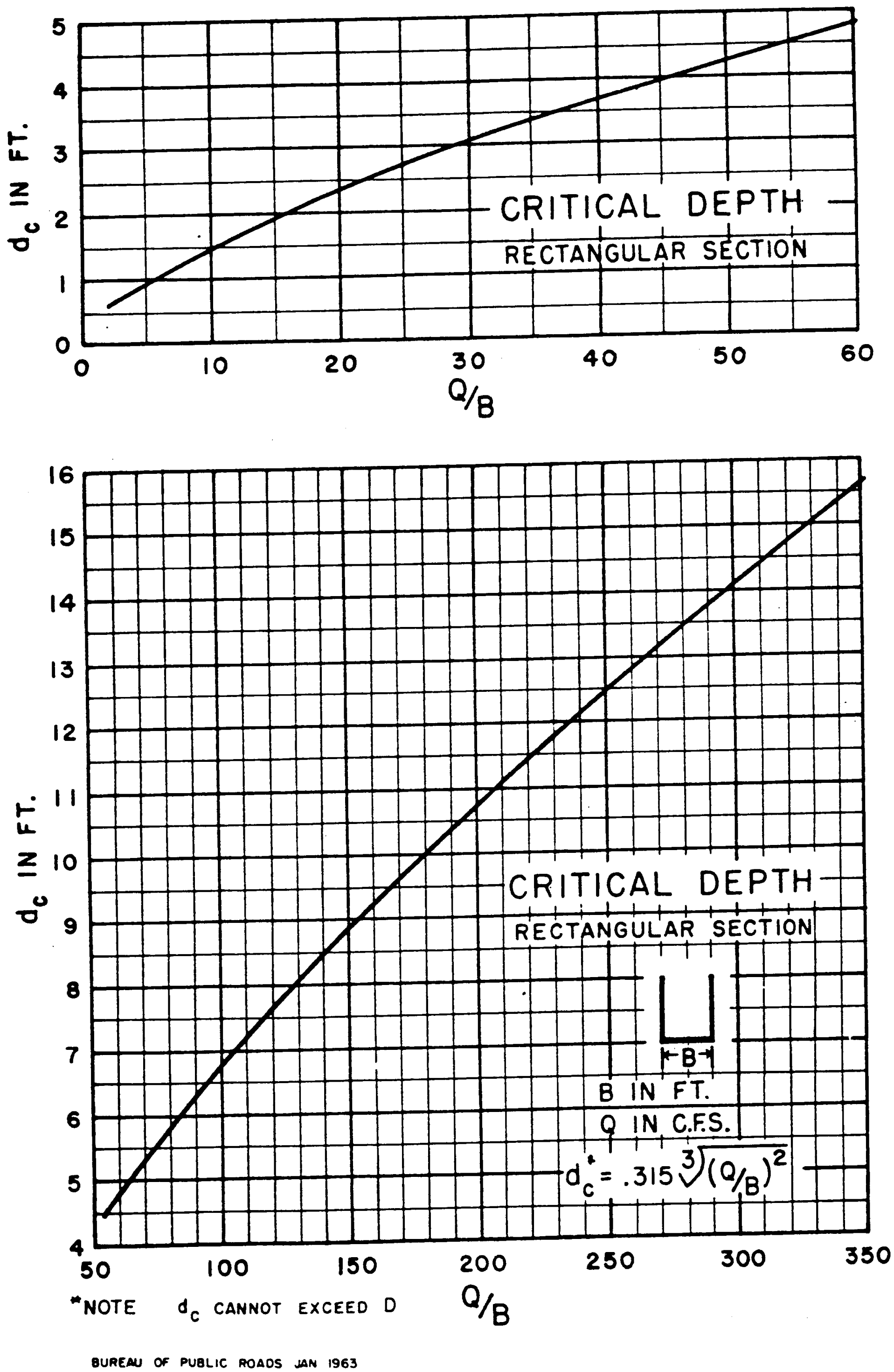


Figure 8.4-6
Critical Depth – Rectangular Section

8.4.2.6 Roadway Overflow

See 8.3.2.6.

8.4.2.7 Outlet Scour and Energy Dissipators

Energy dissipating devices are used where it is desirable to reduce the discharge velocity by inducing high energy losses at the inlet or discharge ends of the structure. They are generally warranted when discharge velocities exceed 14 feet per second.

Energy losses may be induced at the culvert entrance with a drop inlet, or at the outlet using energy dissipating devices and stilling basins to form a hydraulic jump.

Drop inlets are used where headroom is limited, and energy dissipating devices and stilling basins at the discharge are used where headroom is not critical.

The use of drop inlets should generally be reserved for areas where channel slopes are steep. Under these conditions drop inlets enable the reduction of culvert grades and in turn lower discharge velocities. When evaluating a site, a drop inlet may also be applicable on drainage ditches, in addition to channels that are normally dry or do not support fish or other aquatic organism habitat of pronounced significance. The use of a drop inlet requires approval from the Bureau of Structures, as well as coordination with the Department of Natural Resources early in project development.

For outlet devices utilizing the hydraulic jump, two conditions must be present for the formation of a hydraulic jump; the approach depth must be less than critical depth (supercritical flow); and the tailwater depth must be deeper than critical depth (subcritical flow) and of sufficient depth to control the location of the hydraulic jump. Where the tailwater depth is too low to cause a hydraulic jump at the desired location, the required depth can be provided by either depressing the discharge apron or utilizing a broad-crested weir at the end of the apron to provide a pool of sufficient depth. The depressed apron method is preferred since there is less scouring action at the end of the apron. The amount of depression is determined as the difference between the natural tailwater depth and the depth required to form a jump.

There are numerous design concepts of energy dissipating devices and stilling basins that may be adapted for energy dissipation to reduce the velocity and avoid scour at the culvert outlet. The more common type of designs are drop inlets, drop outlets, hydraulic jump stilling basins and riprap stilling basins.

More discussion on energy dissipators for culverts is available in 8.5 references (19), (20), (21), and (22). The designer is strongly advised to closely examine and study reference (20). More detailed discussions about the various types of energy dissipators and their designs are presented in that reference.

8.4.2.7.1 Drop Inlet.

In drop inlet design, flow is controlled at the inlet crest by the weir effect of the drop opening. Drop inlet culverts operate most satisfactorily when the height of drop is sufficient to permit considerable submergence of the culvert entrance without submerging the weir or exceeding limiting headwater depths.

Referring to Figure 8.4-7, the general formula for flow into the horizontal drop opening is:

$$Q = C_1 (2g)^{1/2} L H^{3/2}$$

Where Q is the discharge in c.f.s., L is the crest length 2B+W, H is the depth of flow plus velocity head, and C_1 is a dimensionless discharge coefficient taken as 0.4275. The formula is expressed in english units as:

$$Q = 3.43 LH^{3/2}$$

and

$$L = Q/(3.43H^{3/2})$$

There are four corrections which have to be multiplied times the discharge coefficient C_1, or times the factor 3.43:

1. Correction for head H/W (Table 8.4-1)
2. Correction for box-inlet shape B/W. (Table 8.4-2)
3. Correction for approach channel width W_c/L (Table 8.4-3).

   Where: W_c = approach channel width = Area/Depth
4. Correction for dike effect X/W (Table 8.4-4)

The size of the culvert should be determined by using the discharge (Q) and not allowing the height of water (HW) to exceed the inlet drop plus the critical depth of the weir which is given as:

$$d_c = [(Q/L)^2/g]^{1/3}$$

When using the hydraulic charts of 8.4.2.5, consider the culvert to have a wingwall flare of 0 degrees (extension of sides).

Sample computations are shown in 8.4.2.7.1.1.

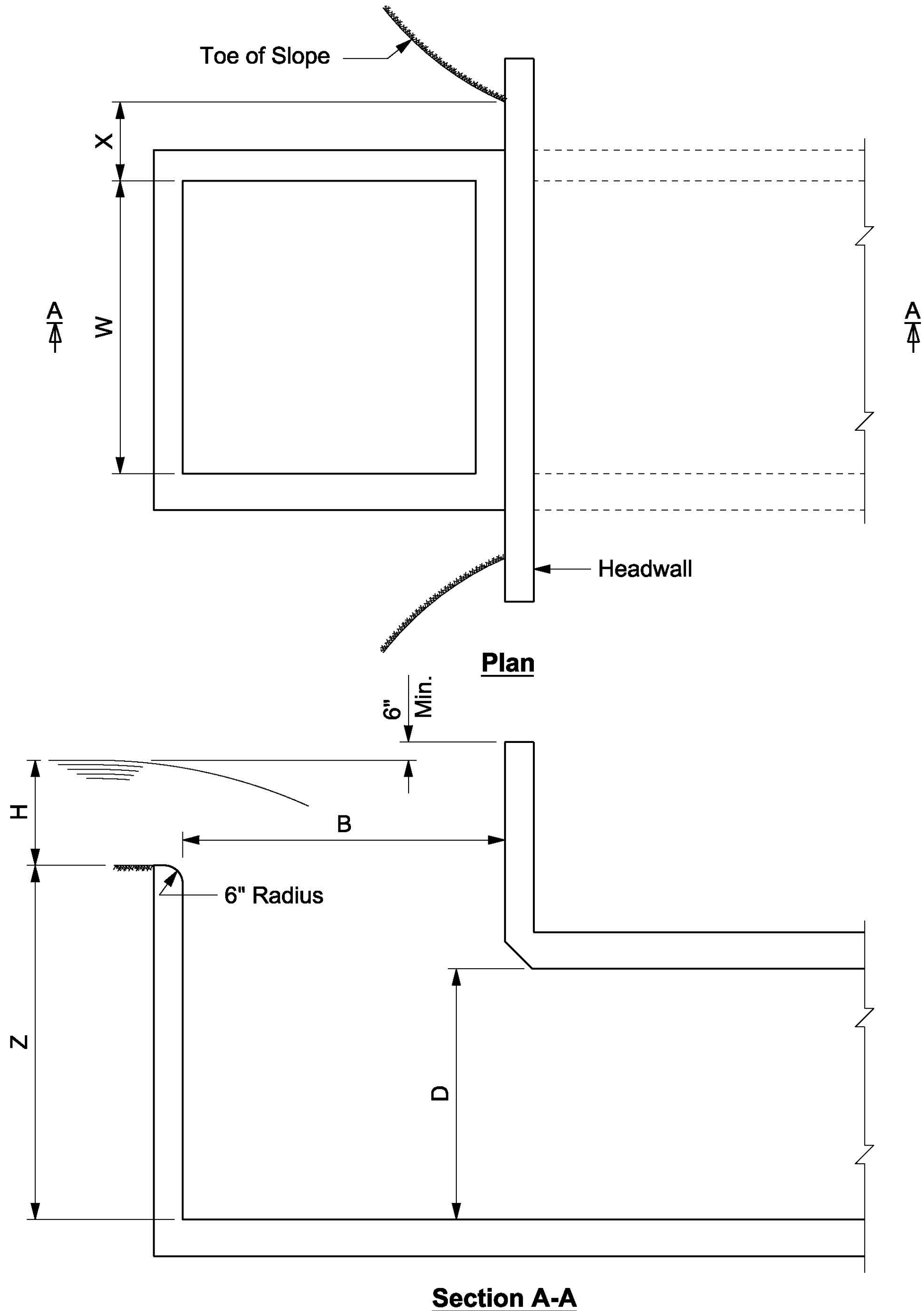


Figure 8.4-7
Box Drop Inlet

H/W	0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0	--	--	--	--	--	0.76	0.8	0.82	0.84	0.86
0.1	0.8	0.88	0.89	0.9	0.91	0.91	0.92	0.92	0.93	0.93
0.2	0.93	0.94	0.94	0.95	0.95	0.95	0.95	0.96	0.96	0.96
0.3	0.97	0.97	0.97	0.97	0.98	0.98	0.98	0.98	0.98	0.98
0.4	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	1
0.5	1	1	1	1	1	1	1	1	1	1
0.6	1	--	--	--	--	--	--	--	--	--
Correction is 1.00 when H/W exceeds 0.6										

Table 8.4-1
Correction for Head
(Control at Box-Inlet Crest)

B/W	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.98	1.01	1.03	1.03	1.04	1.04	1.03	1.02	1.01	1.01
1	1	0.99	0.99	0.98	0.98	0.98	0.97	0.97	0.96	0.96
2	0.96	0.96	0.95	0.95	0.95	0.95	0.95	0.95	0.94	0.94
3	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.93	0.93
4	0.93	--	--	--	--	--	--	--	--	--

Table 8.4-2
Correction for Box-Inlet Shape
(Control at Box-Inlet Crest)

Wc/L	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0.09	0.18	0.27	0.35	0.44	0.53	0.62	0.71	0.8
1	0.84	0.87	0.9	0.92	0.93	0.94	0.95	0.96	0.97	0.97
2	0.98	0.98	0.99	0.99	0.99	0.99	1	1	1	1
3	1	--	--	--	--	--	--	--	--	--
Correction is 1.00 when Wc/L exceeds 3.0										

Table 8.4-3
Correction for Approach-Channel Width
(Control at Box-Inlet Crest)

B/W	X/W						
	0	0.1	0.2	0.3	0.4	0.5	0.6
0.5	0.9	0.96	1	1.02	1.04	1.05	1.05
1	0.8	0.88	0.93	0.96	0.98	1	1.01
1.5	0.76	0.83	0.88	0.92	0.94	0.96	0.97
2	0.76	0.83	0.88	0.92	0.94	0.96	0.97

Table 8.4-4
Correction for Dike Effect
(Control at Box-Inlet Crest)

8.4.2.7.1.1 Drop Inlet Example Calculations

Given:

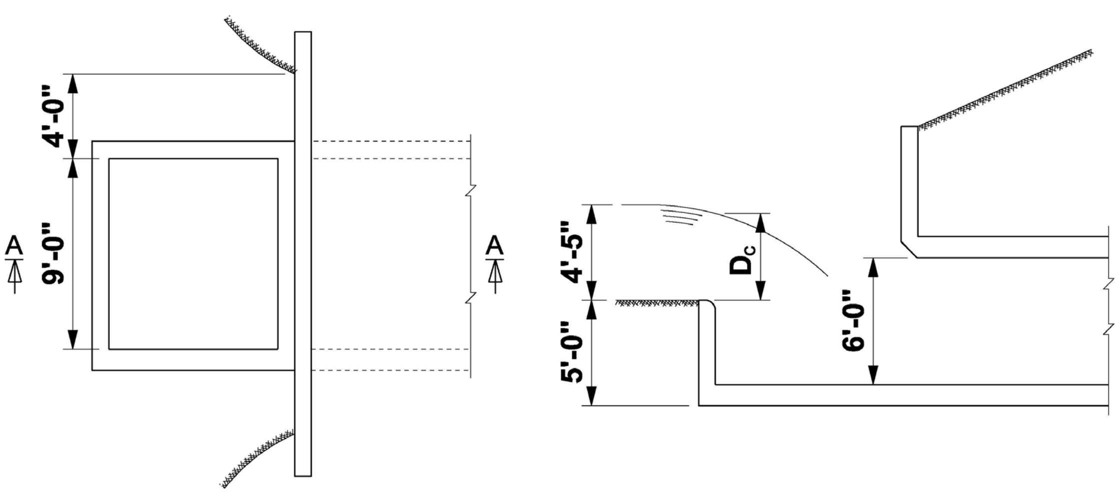
Q = 420 cfs through single 9'x6' box

H = 4.4' in a 27 ft. wide channel

Drop = 5 ft

Assume:

B = $\frac{W}{2} = 4.5$



Plan **Section A-A**

Figure 8.4-8
Drop Inlet Example

Control at inlet crest: $L = \dfrac{Q}{3.43 \cdot H^{3/2}}$

Corrections:

1. $\dfrac{H}{W} = \dfrac{4.4}{9} = 0.49 \Rightarrow 1.00$

2. $\dfrac{B}{W} = \dfrac{4.5}{9} = 0.5 \Rightarrow 1.04$

3. $\dfrac{W_c}{L} = \dfrac{27}{9 + 2(4.5)} = \dfrac{27}{18} = 1.50 \Rightarrow 0.94$

4. $\dfrac{X}{W} = \dfrac{4.0}{9.0} = 0.44 \Rightarrow 1.04$

Total Correction = 1.00 x 1.04 x 0.94 x 1.04 = 1.02

$$L = \frac{420}{1.02 \cdot 3.43 \cdot 4.4^{3/2}} = \frac{420}{1.02 \cdot 3.43 \cdot 9.23} = 13.01 < (2B + W) = 18 \Rightarrow OK$$

$$d_c = \sqrt[3]{\frac{Q^2}{L^2 g}} = \left(\frac{17.64x10^4}{3.24 \cdot 3.22x10^3}\right)^{1/3} = 16.85^{1/3} = 2.56$$

HW must be less than Z+d_c to prevent submerged weir. With inlet control, from Figure 8.4-3:

$$\frac{HW}{D} = 1.19$$

HW = 1.19x6 = 7.14

7.14 < (5+2.56) = 7.56, therefore weir controls

8.4.2.7.2 Drop Outlets

This generalized design is applicable to relative heights of fall ranging from 1.0 y/d_c to 15 y/d_c and to crest lengths greater than 1.5 d_c. Here y is the vertical distance between the crest and the stilling basin floor and d_c is the critical depth of flow.

$d_c = 0.315[(Q/B)^2]^{1/3}$

Referring to Figure 8.4-10 and Figure 8.4-9, this design uses the following formulas:

1. The minimum length L_b of the stilling basin is:

$X_a + X_b + X_c = X_a + 2.55\ d_c$

a. The distance X_a from the headwall to the point where the surface of the upper nappe strikes the stilling basin floor is solved graphically in Figure 8.4-9.

b. The distance X_b from the point at which the surface of the upper nappe strikes the stilling basin floor to the upstream face of the floor blocks is:

$X_b = 0.8\ d_c$

c. The distance X_c, between the upstream face of the floor blocks and the end of the stilling basin is:

$X_c \geq 1.75\ d_c$

2. The floor blocks are proportioned as follows:

a. The height of the floor blocks is:

$0.8\ d_c$

b. The width and spacing of the floor blocks are approximately:

$0.4\ d_c$

A variation of $\pm$ 0.15 d_c from this limit is permissible.

c. The floor blocks are square in plan.

d. The floor blocks occupy between 50 and 60 percent of the stilling basin width.

3. The height of the end sill is:

$0.4\ d_c$

4. The sidewall height above the tailwater level is:

$0.85\ d_c$

5. The minimum height d_2, of the tailwater surface above the floor of the stilling basin is:

$d_2 = 2.15\ d_c$

In cases where the approach velocity head is greater than 1/3 of the specific head (velocity head + elevation head), X_a is checked by the formula below and the greater X_a value is used.

$$X_a^{\ 2} = \left(\frac{2 \cdot V^2}{g} \right) \cdot y_1$$

Where:

y_1 = top of water at crest

V = velocity of approach

Sometimes high values of d_c become unworkable, resulting in a need for large drops, high end sills and floor blocks. To prevent this d_c may be reduced by flaring the end of the barrel. The flare angle is approximately 150/V where V is the velocity at the beginning of the taper.

Sample computations are shown in 8.4.2.7.2.1.

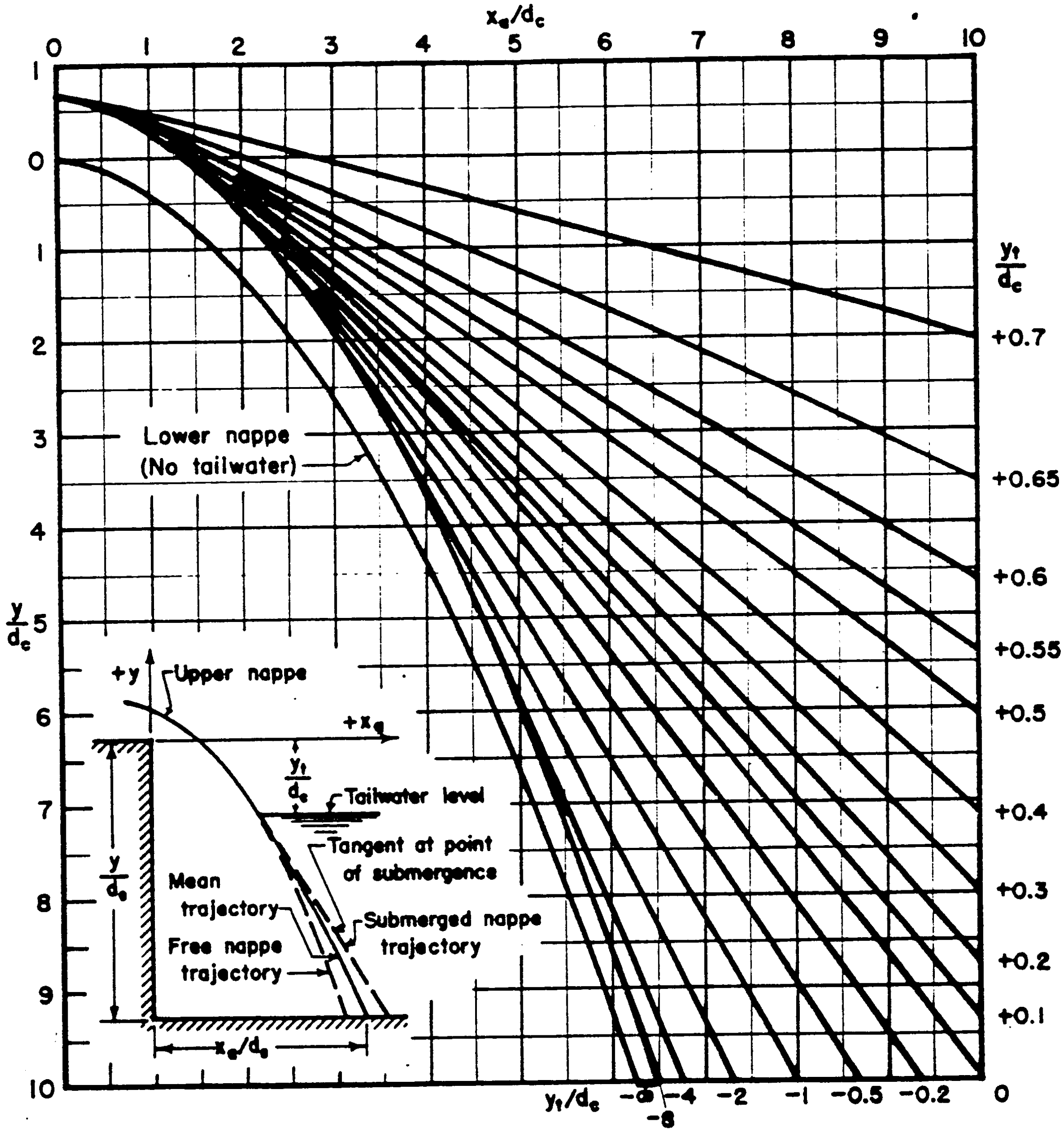


Figure 8.4-9

Design Chart for Determination of "X_a"

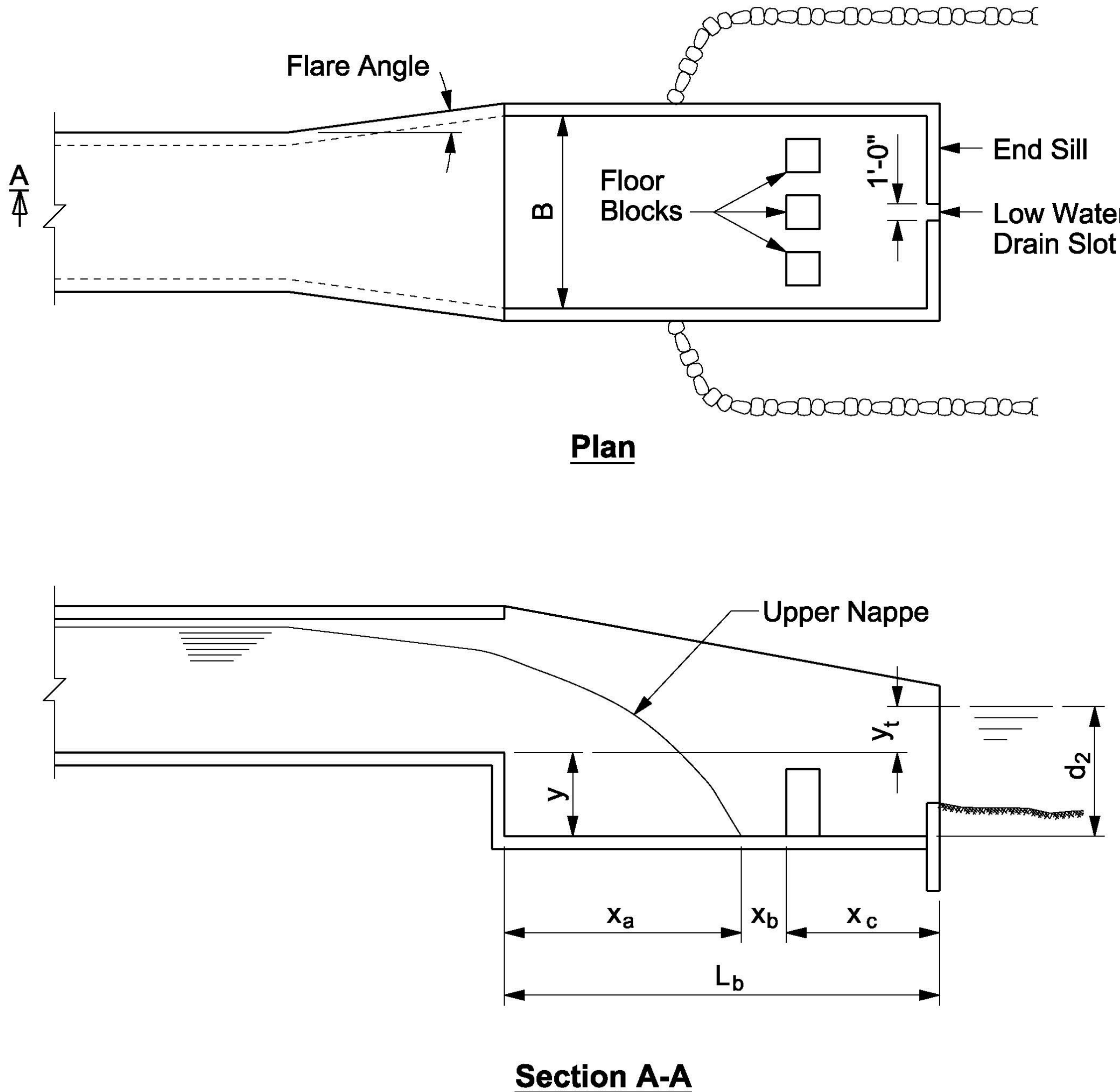


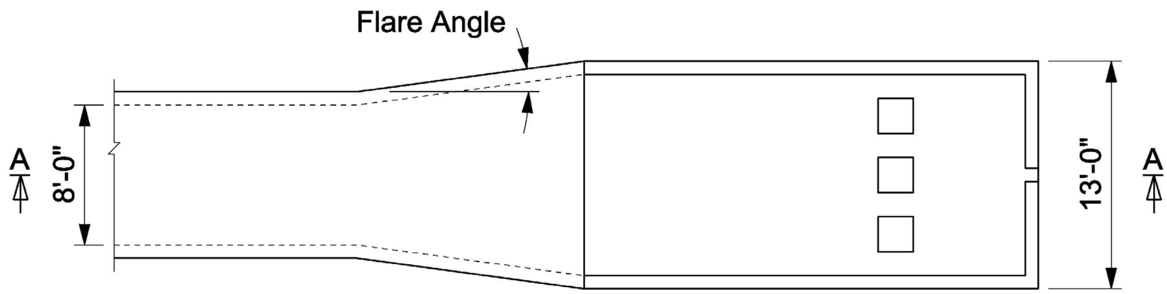
Figure 8.4-10
Straight Drop Outlet Stilling Basin

8.4.2.7.2.1 Drop Outlet Example Calculations

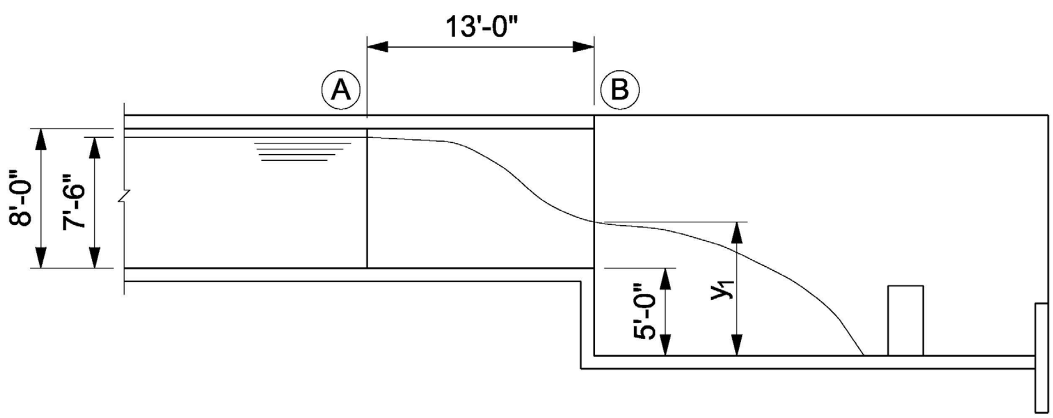
Given:

Q	=	800 cfs through single 8'x8' box
V	=	13.5 fps in the box
Drop	=	5 ft

Depth = 7.5 ft



Plan



Section A-A

Figure 8.4-11
Drop Outlet Example

Assumptions:

- That the specific head of "A" is approximately equal to the specific head at "B". Therefore, the elevation head + velocity head at "A" = elevation head + velocity head at "B".

- The end sill height should be less than or equal to 2'-0".

If the drop were placed at "A":

$$d_c = 0.315 \cdot \sqrt[3]{\left(\frac{Q}{B}\right)^2} = 0.315 \cdot (100)^{2/3} = 6.78$$

And end sill = 0.4dc = 2'-9" which exceeds 2'-0, therefore flare outlet.

To obtain a 2'-0" sill, set d_c = 2'-0"/0.4 = 5 ft

$$B = \left(\frac{0.315 \cdot Q^{2/3}}{d_c}\right)^{3/2} = \left(\frac{0.315 \cdot 800^{2/3}}{5}\right)^{3/2} = 13'$$

Flare from B = 9 ft to B = 13 ft at an angle of 150/13.5 = 11^0

$$\text{Length} = \frac{\left(\frac{13-8}{2}\right)}{\tan 11^0} = 13'$$

$$\text{Specific Head, } H_A = 7.5 + \frac{V_A{}^2}{2g} = \frac{13.5^2}{2 \cdot 32.2} = 10.33'$$

By trial and error; assume $\frac{V_B{}^2}{2g} = 7.5'$

$$V_B = (2 \cdot 32.2 \cdot 7.5)^{1/2} = 22\text{fps}$$

Elevation head (depth) = 10.33-7.2 = 2.83'

Check trial; Q = AV = (13x2.83)x22 = 809 cfs, Q_{actual} = 800 cfs, OK

$$d_c = 0.315 \cdot \sqrt[3]{\left(\frac{Q}{B}\right)^2} = 0.315 \cdot \left(\frac{800}{13}\right)^{2/3} = 0.315 \cdot 15.6 = 4.91'$$

$$\frac{h_v}{H} = \frac{\left(\frac{V_B{}^2}{2g}\right)}{10.33} = \frac{7.5}{10.33} = 0.725 > \frac{1}{3} \qquad \therefore X_a{}^2 = \frac{2V^2}{g} y_1$$

$$X_a = \left[\frac{2 \cdot 22^2 \cdot (5 + 2.83)}{32.2}\right]^{1/2} = 15.35' \qquad \text{Use } X_a = 15'\text{-}6''$$

Dimensions:

Height of floor blocks	=	0.8 x 4.91 = 4'-0"
Height of end sill	=	0.4 x 4.91 = 2'-0"
Length of Basin	=	15.5+2.55 d_c = 28'
Floor Blocks	=	2'-0" square

Height of Sidewalls = $(2.15 + 0.85)d_c$ = 14.48' above basin floor. Use 13'-0"

8.4.2.7.3 Hydraulic Jump Stilling Basins

The simplest form of a hydraulic jump stilling basin has a straight centerline and is of uniform width. A sloping apron or a chute spillway is typically used to increase the Froude number as the water flows from the culvert to the stilling basin. The outlet barrel of the culvert is also sometimes flared to decrease y_1 so that the tailwater elevation necessary to cause a hydraulic jump need not be so high. This is done using the 150/V relationship as in the drop outlet sample problem. y_1 is usually kept in the 2-3 foot range.

Referring to Figure 8.4-12, the required tailwater is computed by the formula:

$$y_2/y_1 = \frac{1}{2}\ [(1+8F_1^2)^{1/2} - 1]$$

Where:

y_2 = tailwater height required to cause the hydraulic jump,

F_1 = Froude number = $v_1 / (gy_1)^{1/2}$

g = acceleration of gravity,

y_1 = velocity at beginning of jump.

End sill height (ΔZ_0) is determined graphically from Figure 8.4-13

Length of jump is assumed to be 6 times the depth change (y_2-y_1).

In many cases the tailwater height isn't deep enough to cause the hydraulic jump. To remedy this, the slope of the culvert may be increased to greater than the slope of the streambed. This will result in an apron depressed such that normal tailwater is of sufficient depth.

The problem of scour on the downstream side of the end sill can be overcome by providing riprap in the stream bottom. If riprap is used, it starts from the top of the sill at a maximum slope of 6:1 up from end sill to original streambed. If no riprap is used, the streambed begins at the top of the end sill.

More detailed discussion about the various types of hydraulic jump stilling basins and their design can be found in 8.5 reference (20).

Sample computations are shown in 8.4.2.7.3.1.

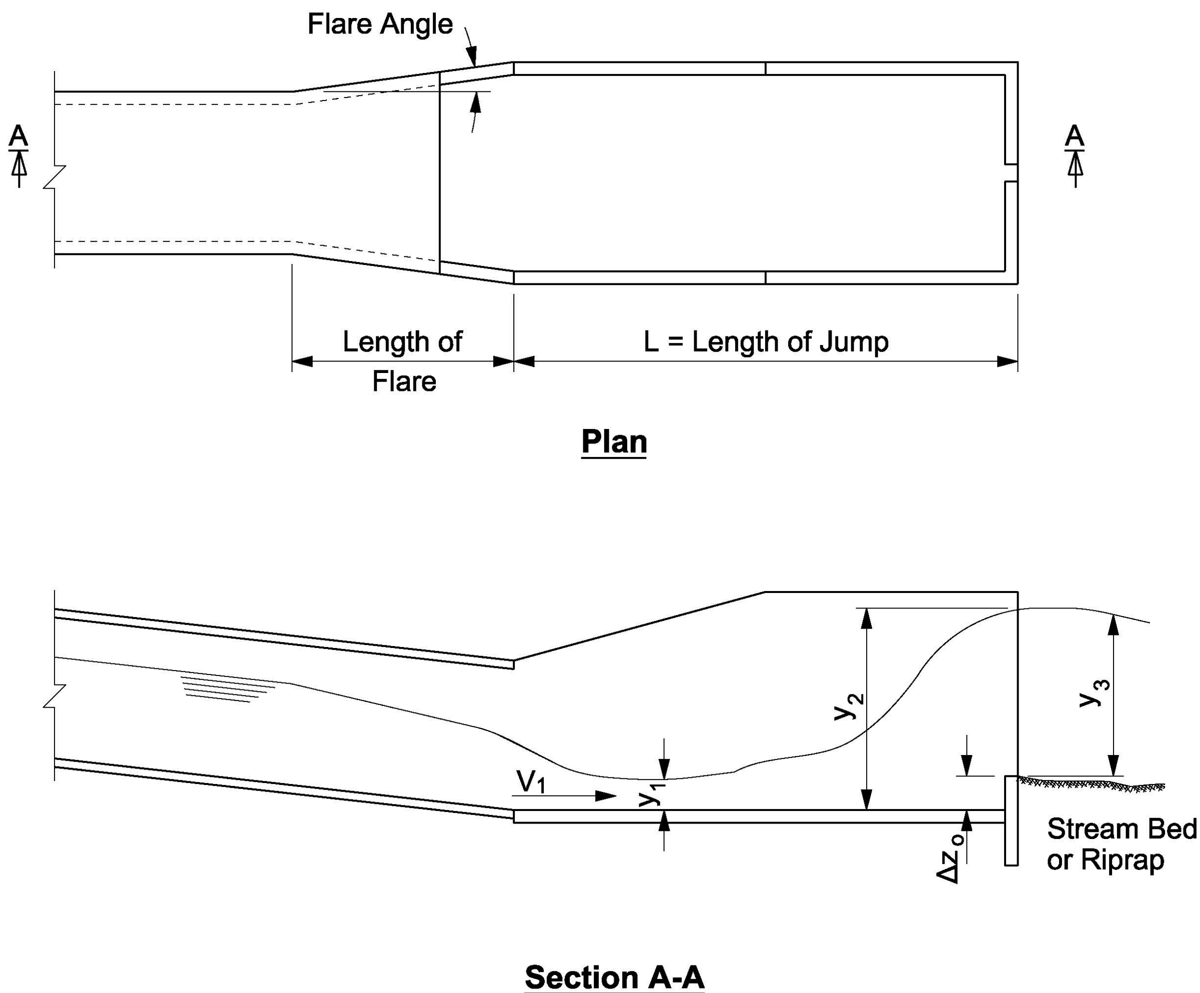


Figure 8.4-12
Hydraulic Jump Stilling Basin

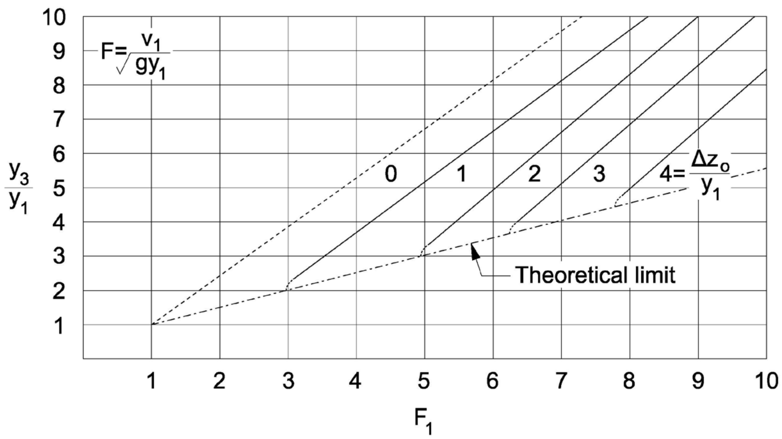


Figure 8.4-13
Characteristics of a Hydraulic Jump at an Abrupt Rise

8.4.2.7.3.1 Hydraulic Jump Stilling Basin Example Calculations

Given:

A discharge of 600 cfs flows through a 7’x6’ box culvert at 16 fps and a depth of 5.8’. Normal tailwater depth in the outlet channel is 5.0 feet.

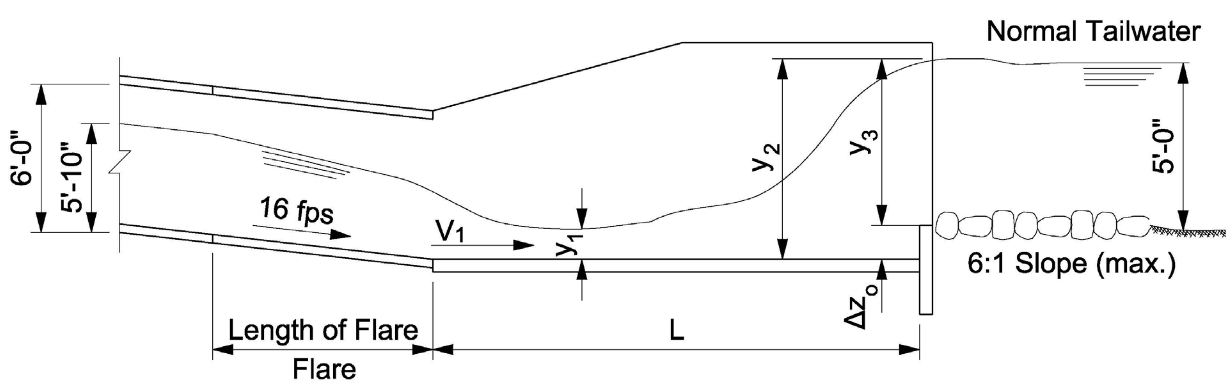


Figure 8.4-14
Hydraulic Jump Stilling Basin Example

Flare of wings $= \dfrac{150}{16} \approx 9^0$

$$H = 5.8 + \frac{16^2}{2 \times 32.2} = 5.8 + 3.975 = 9.775$$

Assume:

$y_1 = 2.2$ and $\frac{V_1^2}{2 \cdot g} = 9.775 - 2.2 = 7.575'$

$V_1 = (2 \times 32.2 \times 7.575)^{1/2} = 22.1$ fps

$Q = 600 = AV = 2.2 \times \text{width} \times 22.1$, width = 12.36

Length of flare = $\frac{\frac{(12.36-7)}{2}}{\tan 9^0} = 17'$

$Y_1 = 2.20$

$V_1 = 22.1$

$$F_1 = \frac{V_1}{\sqrt{g \cdot y_1}} = \frac{22.1}{\sqrt{32.2 \times 2.2}} = 2.63$$

$$y_2 = y_1 \cdot \frac{1}{2} \cdot \left(\sqrt{1 + 8 \times 2.63^2} - 1\right) = 7.15$$

$L = 6(y_2 - y_1) = 6\,(7.15 - 2.20) = 29.7'$ use L = 30 ft.

Assume $y_3 = 5'$

$y_3/y_1 = 5/2.2 = 2.27$

From Figure 8.4-13, $\Delta Z_o/y_1 = 0.5$

$\Delta Z_o = 1.1$, use 1'-6"

8.4.2.7.4 Riprap Stilling Basins

The riprap stilling basins, in many cases, is a very economical approach to dissipate energy at culvert outlets and avoid damaging scour. A good treatise on riprap stilling basin is given in the FHWA Hydraulic Design of Energy Dissipators for Culverts and Channels, see 8.5 reference (20).

8.4.2.8 Select Culvert Design Alternatives

The "proposed culvert" design shall be based on several design factors. In most design situations, the pertinent hydraulic factors discussed in 8.4.1 will dictate the final selection of culvert size, length, scour protection, as well as the approach roadway design.

8.5 References

1. Wisconsin Department of Natural Resources, *Wisconsin's Floodplain Management Program, Chapter NR116*, Register, August 2004, No. 584.

2. U. S. Geological Survey, *Flood-Frequency Characteristics of Wisconsin Streams*. Water-Resources Investigations Report 03-4250, 2003. This report can be found on the USGS web site using the following link:

   http://wi.water.usgs.gov/publications/flood/currentreport.html

3. U. S. Geological Survey, *Guidelines for Determining Flood Flow Frequency, Bulletin #17B* Revised September 1981, Editorial Corrections, March 1982.

4. U.S. Department of Agriculture, Soil Conservation Service, Urban Hydrology for Small Watersheds, Technical Release 55 (2nd Edition), June 1986.

5. Ven Te Chow, Ph.D. *Open Channel Hydraulics* (New York, McGraw-Hill Book Company 1959).

6. U.S. Department of Transportation, Federal Highway Administration, *Design Charts for Open-Channel Flow Hydraulic Design,* Series No. 3, August 1961.

7. U.S. Army Corps of Engineers, *HEC-RAS River Analysis System Users Manual*, (CPD-68), Hydrologic Engineering Center, Davis CA, Version 4.0, March 2008.

8. U.S. Army Corps of Engineers, *HEC-RAS River Analysis System Hydraulic Reference Manual* (CPD-69), Hydrologic Engineering Center, Davis CA, Version 4.0, March 2008.

9. U.S. Army Corps of Engineers, *HEC-RAS River Analysis System Applications Guide* (CPD-70), Hydrologic Engineering Center, Davis CA, Version 4.0, March 2008.

10. U.S. Department of Interior, Geological Survey, *Measurement of Peak Discharge at Width Contractions by Indirect Methods; Techniques of Water-Resources Investigation of the U.S.G.S.*, Chapter A4, Book 3, Third printing 1976.

11. L.A. Arneson and J.O. Shearman, *User's Manual for WSPRO-A computer Model for Water Surface Profile Computations,* FHWA Report No. FHWA-SA-98-080, June 1998.

12. J.O. Shearman, W. H. Hirby, V.R. Schneider, H.N. Flippo, *Bridge Waterways Analysis Model*, Research Report, FHWA Report No. FHWO-RD-86/108.

13. U.S. Department of Transportation, FHWA, Hydraulic Design Series (HDS), Number 5, *Hydraulic Design of Highway Culverts*, September 2001, Revised May 2005.

14. U.S Department of Transportation, Federal Highway Administration, *Hydraulic Engineering Circular No. 18, Evaluating Scour at Bridges*, 4th Edition, May 2001.

15. U.S Department of Transportation, Federal Highway Administration, *Hydraulic Engineering Circular No. 20, Stream Stability at Highway Structures*, 3rd Edition, March 2001.

16. U.S. Department of Transportation, Federal Highway Administration, *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*, Office of Engineering, Bridge Division, Report No. FHWA-PD-96-001, December 1995.

17. U.S. Department of Transportation, Federal Highway Administration, *Highways in the River Environment,* Report No. FHWA-HI-90-016, February 1990.

18. U.S. Department of Transportation, FHWA, *Debris-Control Structures, Evaluation and Countermeasures, Third Edition,* Hydraulic Engineering Circular (HEC) No.9, Publication No. FHWA-IF-014-016, October 2005.

19. U.S. Department of Interior, Bureau of Reclamation, *Design of Small Dam*, 3rd Edition Washington D.C. 1987.

20. U.S. Department of Transportation, Federal Highway Administration, *Hydraulic Design of Energy Dissipators for Culverts and Channels*, Hydraulic Engineering Circular (HEC) No. 14, Third Edition, Publication No. FHWA-NHI-06-086, July 2006.

21. Blaisdell, Fred W. and Donnelly, Charles A., *Hydraulic Design of the Box Inlet Drop Spillway*, U.S. Department of Agriculture, Soil Conservation Service, SCS-TP-106, July, 1951.

22. Blaisdell, Fred W. and Donnelly, Charles A., *Straight Drop Spillway Stilling Basin*, University of Minnesota, St. Anthony Falls Hydraulic Laboratory, November, 1954.

8.6 Appendix 8-A, Check List for Hydraulic/Site Report

A hydraulic and site report shall be prepared for all stream crossing bridge and culvert projects that are completed by consultants. The report shall be submitted to the Bureau of Structures for review along with the “Stream Crossing Structure Survey Report” and preliminary structure plans (see WisDOT Bridge Manual, 6.2.1). The hydraulic and site report needs to include information necessary for the review of the hydraulic analysis and the type, size and location of proposed structure. The following is a list of the items that need to be included in the hydraulic site report:

- Document the location of the stream crossing or project site. Indicate county, municipality, Section, Town, and Range.

- List available information and references for methodologies used in the report. Indicate when survey information was collected and what vertical datum was used as reference for elevations used in hydraulic models and shown on structure plans. Indicate whether the site is in a mapped flood hazard area and type of that mapping, if any.

- Provide complete description of the site, including description of the drainage basin, river reach upstream and downstream of the site, channel at site, surrounding bank and over bank areas, and gradient or slope of the river. Also, provide complete description of upstream and downstream structures.

- Provide a summary discussion of the magnitude and frequency of floods to be used for design. Hydrologic calculations shall be provided to the Bureau of Structures beforehand for their review and concurrence. Indicate in the hydraulic site report when calculations were submitted and whether approval was obtained.

- Provide a description of the hydraulic analyses performed for the project. Indicate what models were used and the basis for and assumptions used in the selection of various modeling parameters. Specifically, discuss the assumptions used for defining the modeling reach boundary conditions, roughness coefficients, location and source of hydraulic cross sections, and any assumptions made in selecting the bridge modeling methodology. (Hydraulic calculations shall be submitted with the hydraulic site report).

- Provide a complete description of the existing structure, including a description of the geometry, type, size and material. Indicate the sufficiency rating of the structure. Provide information about observed scour, flooding, roadway overtopping, ice or debris, navigation clearance and any other structurally or hydraulically pertinent information. Provide a discussion of calculated hydraulic characteristics at the site.

- Provide a description of the various sizing constraints considered at the site, including but not limited to regulatory requirements, hydraulic and roadway geometric conditions, environmental and constructability considerations, etc.

- Provide a discussion of the alternatives considered for this project including explanations of how certain alternatives are removed from consideration and how the recommended alternative is selected. Include a cost comparison.

- Provide complete description of proposed structure including calculated hydraulic characteristics.

- Provide a discussion of calculated scour depths, recommended scour prevention measures and assigned scour code. (Scour calculations shall be submitted with the hydraulic site report).

- Provide a summary table comparing calculated hydraulic characteristics for existing and proposed conditions.

8.7 Appendix 8-B, FHWA Hydraulic Engineering Publications

Note: Some links may be obsolete, but will be updated in the future.

Code	Title	Year	Publication #	NTIS #
HDS 01	Hydraulics of Bridge Waterways	1978	FHWA-EPD-86-101	PB86-181708
HDS 02	Highway Hydrology Second Edition	2002	FHWA-NHI-02-001	
HDS 03	Design Charts for Open-Channel Flow	1961	FHWA-EPD-86-102	PB86-179249
HDS 04	Introduction to Highway Hydraulics	2001	FHWA-NHI-01-019	
HDS 05	Hydraulic Design of Highway Culverts	2005	FHWA-NHI-01-020	
HDS 06	River Engineering for Highway Encroachments	2001	FHWA-NHI-01-004	
HEC 09	Debris Control Structures Evaluation and Countermeasures	2005	FHWA-IF-04-016	
HEC 11	Design of Riprap Revetment	1989	FHWA-IP-89-016	PB89-218424
HEC 14	Hydraulic Design of Energy Dissipators for Culverts and Channels	2006	FHWA-NHI-06-086	
HEC 15	Design of Roadside Channels with Flexible Linings, Third Edition	2005	FHWA-IF-05-114	
HEC 17	The Design of Encroachments on Flood Plains Using Risk Analysis	1981	FHWA-EPD-86-112	PB86-182110
HEC 18	Evaluating Scour at Bridges, Fourth Edition	2001	FHWA-NHI-01-001	
HEC 18	Evaluating Scour at Bridges, Fourth Edition (Errata Sheet)	2001		
HEC 20	Stream Stability at Highway Structures Third Edition	2001	FHWA-NHI-01-002	
HEC 20	Stream Stability at Highway Structures Third Edition (Errata Sheet)	2001	FHWA-NHI-01-002	
HEC 21	Bridge Deck Drainage Systems	1993	FHWA-SA-92-010	PB94-109584
HEC 22	Urban Drainage Design Manual Second Edition	2001	FHWA-NHI-01-021	
HEC 23	Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance Second Edition	2001	FHWA-NHI-01-003	
HEC 23	Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance Second Edition (Errata Sheet)	2001		
HEC 24	Highway Stormwater Pump Station Design (cover)	2001	FHWA-NHI-01-007	
HEC 24	Highway Stormwater Pump Station Design	2001	FHWA-NHI-01-007	
HEC 25	Tidal Hydrology, Hydraulics, and Scour at Bridges	2004	FHWA-NHI-05-077	
HEC 25	Highways in the Coastal Environment - 2nd edition	2008	FHWA-NHI-07-096	
HRT	Assessing Stream Channel Stability at Bridges in Physiographic Regions	2006	FHWA-HRT-05-072	

Code	Title	Year	Publication #	NTIS #
HRT	Effects of Inlet Geometry on Hydraulic Performance of Box Culverts	2006	FHWA-HRT-06-138	
HRT	Junction Loss Experiments: Laboratory Report	2007	FHWA-HRT-07-036	
HRT	Hydraulics Laboratory Fact Sheet	2007	FHWA-HRT-07-054	
Other	Geosynthetic Design and Construction Guidelines	1995	FHWA-HI-95-038	PB95-270500
Other	Underwater Evaluation And Repair of Bridge Components	1998	FHWA-DP-98-1	
Other	Best Management Practices for Erosion and Sediment Control	1995	FHWA-FLP-94-005	
Other	Underwater Inspection of Bridges	1980	FHWA-DP-80-1	
Other	Culvert Management Systems User Manual	2001	FHWA-02-001	
Other	FHWA Hydraulics Library on CD-ROM FHWA Hydraulics Library on CD-ROM (Updated Browser)	2002		
Other	Hydraulic Performance of Curb and Gutter Inlets	1999	FHWA-KU-99-1	
Other	Culvert Management Systems Source Code	2001		
Other	NCHRP Report 25-25 (04) Environmental Stewardship Practices, Procedures, and Policies for Highway Construction and Maintenance	2004		
Other	New England Transportation Consortium: Performance Specs for Wood Waste Materials as an Erosion Control Mulch and as a Filter Berm	2001	FHWA-NETC 25	
Other	Bridge Scour Protection Systems Using Toskanes	1994	FHWA-PA-94-012	PB95-266318
Other	Structural Design Manual for Improved Inlets and Culverts	1983	FHWA-IP-83-6	PB84-153485
Other	Culvert Inspection Manual	1986	FHWA-IP-86-2	PB87-151809
RD	Bottomless Culvert Scour Study: Phase II Laboratory Report	2007	FHWA-HRT-07-026	
RD	Effects of Gradation and Cohesion on Scour, Volume 2, "Experimental Study of Sediment Gradation and Flow Hydrograph Effects on Clear Water Scour Around Circular Piers"	1999	FHWA-RD-99-184	PB2000-103271
RD	Effects of Gradation and Cohesion on Scour, Volume 1, "Effect of Sediment Gradation and Coarse Material Fraction on Clear Water Scour Around Bridge Piers"	1999	FHWA-RD-99-183	PB2000-103270
RD	Portable Instrumentation for Real Time Measurement of Scour At Bridges	1999	FHWA-RD-99-085	PB2000-102040
RD	Users Primer for BRI-STARS	1999	FHWA-RD-99-191	PB2000-107371

Code	Title	Year	Publication #	NTIS #
RD	Effects of Gradation and Cohesion on Scour, Volume 3, "Abutment Scour for Nonuniform Mixtures"	1999	FHWA-RD-99-185	PB2000-103272
RD	Remote Methods of Underwater Inspection of Bridge Structures	1999	FHWA-RD-99-100	PB9915-7968
RD	Hydraulics of Iowa DOT Slope-Tapered Pipe Culverts	2001	FHWA-RD-01-077	
RD	Users Manual for BRI-STARS	1999	FHWA-RD-99-190	PB2000-107372
RD	Effects of Gradation and Cohesion on Scour, Volume 4, "Experimental Study of Scour Around Circular Piers in Cohesive Soils"	1999	FHWA-RD-99-186	PB2000-103273
RD	Effects of Gradation and Cohesion on Scour, Volume 5, "Effect of Cohesion on Bridge Abutment Scour"	1999	FHWA-RD-99-187	PB2000-103274
RD	Effects of Gradation and Cohesion on Scour, Volume 6, "Abutment Scour in Uniform and Stratified Sand Mixtures"	1999	FHWA-RD-99-188	PB2000-103275
RD	Durability Analysis of Aluminized Type 2 Corrugated Metal Pipe	2000	FHWA-RD-97-140	
RD	Performance Curve for a Prototype of Two Large Culverts in Series Dale Boulevard, Dale City, Virginia	2001	FHWA-RD-01-095	
RD	Bottomless Culvert Scour Study: Phase I Laboratory Report	2002	FHWA-RD-02-078	
RD	Bridge Scour in Nonuniform Sediment Mixtures and in Cohesive Materials: Synthesis Report	2003	FHWA-RD-03-083	PB-2204-104690
RD	Enhanced Abutment Scour Studies For Compound Channels	2004	FHWA-RD-99-156	
RD	Field Observations and Evaluations of Streambed Scour at Bridges	2005	FHWA-RD-03-052	
RD	South Dakota Culvert Inlet Design Coefficients	1999	FHWA-RD-01-076	

Figure 8.7-1

FHWA Hydraulic Engineering Publications

FHWA Hydraulics Engineering Software		
Software	Title	Year
HY 7	Bridge Waterways Analysis Model (WSPRO)	2005
HY 7	WSPRO User's Manual (Version 061698) (pdf 2.1 MB)	1998
HY 8	Culvert Hydraulic Analysis Program, Version 7.0	2007
HDS 5	HDS 5 Hydraulic Design of Highway Culverts (pdf, 9.25 mb)	2001
HDS 5	HDS 5 Chart Calculator	2001
HY 11	Preliminary Analysis System for WSP	1989
HY 11	PAS USERS MANUAL (ISDDC)	1989
HY 11	Accuracy of Computed Water Surface Profiles (ISDDC)	1986
FESWMS	FESWMS (Version 3.1.5)	2003
FESWMS	FESWMS User's Manual	2003
HY 22	Visual Urban	2002
HY 22	HEC 22 - Urban Drainage Manual	2001
BRI-STARS	Bridge Stream Tube for Alluvial River Sim	2000
BRI-STARS	BRI-STARS Users Manual	2000
HYRISK	HYRISK Setup (zip, 13 mb)	2002
Hydraulics Software by Others		
Software	Title	Year
BCAP	Broken-back Culvert Analysis Program (Version 3.0)	2002
CAESAR	Cataloging And Expert evaluation of Scour risk And River stability at bridge sites	2001
CHL	Coastal & Hydraulics Laboratory USACE	
FishXing	Fish Passage through Culverts USFS	
HEC	Hydrologic Engineering Center USACE	
HyperCalc	HyperCalc Plus	2002
NSS	National Streamflow Statistics Program	
PEAKFQ	PEAKFQ	1995
SMS	Surface-Water Modeling System (SMS)	2001
StreamStats	StreamStats	
USGS	Water Resources Applications Software USGS	
WMS	Watershed Modeling System (WMS)	

Figure 8.7-2
FHWA Hydraulics Software List

Table of Contents

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:

Pier configurations shall be determined by providing the most efficient cast-in-place concrete pier design, unless approved otherwise. See 7.1.4.1.2 for policy guidance. 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, except when used for pile encased piers. These requirements are intended to guard against the effects of scour.

13.1.3 Pier Construction

Except as allowed for pile encased piers (see 13.2.3) 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.

13.2 Pier Types

The pier types most frequently used in Wisconsin are:

- Multi-column piers (Standards for Multi-Columned Pier and for Multi-Columned Pier – Type 2)
- Pile bents (Standard for Pile Bent)
- Pile encased piers (Standard for Pile Encased Pier)
- Solid single shaft / hammerheads (Standards for Hammerhead Pier and for Hammerhead Pier – Type 2)

Design loads shall be calculated and applied to the pier in accordance with 13.4 and 13.5. The following sections discuss requirements specific to each of the four common pier types.

13.2.1 Multi-Column Piers

Multi-column piers, as shown in Standard for Multi-Columned Pier, are the most commonly used pier type for grade separation structures. Refer to 13.6 for analysis guidelines.

A minimum of three columns shall be provided to ensure redundancy should a vehicular collision occur. If the pier cap cantilevers over the outside columns, a square end treatment is preferred over a rounded end treatment for constructability. WisDOT has traditionally used round columns. Column spacing for this pier type is limited to a maximum of 25'.

Multi-column piers are also used for stream crossings. They are especially suitable where a long pier is required to provide support for a wide bridge or for a bridge with a severe skew angle.

Continuous or isolated footings may be specified for multi-column piers. The engineer should determine estimated costs for both footing configurations and choose the more economical configuration. Where the clear distance between isolated footings would be less than 4'-6", a continuous footing shall be specified.

A variation of the multi-column pier in Standard for Multi-Columned Pier is produced by omitting the cap and placing a column under each girder. This detail has been used for steel girders with girder spacing greater than 12'. This configuration is treated as a series of single column piers. The engineer shall consider any additional forces that may be induced in the superstructure cross frames at the pier if the pier cap is eliminated. The pier cap may not be eliminated for piers in the floodplain, or for continuous slab structures which need the cap to facilitate replacement of the slab during future rehabilitation.

See Standard for Highway Over Railroad Design Requirements for further details on piers supporting bridges over railways.

13.2.2 Pile Bents

Pile bents are most commonly used for small to intermediate stream crossings and are shown on the Standard for Pile Bent.

Pile bents shall not be used to support structures over roadways or railroads due to their susceptibility to severe damage should a vehicular collision occur.

For pile bents, pile sections shall be limited to 12¾" or 14" diameter cast-in-place reinforced concrete piles with steel shells spaced at a minimum center-to-center spacing of 3'. A minimum of five piles per pier shall be used on pile bents. When a satisfactory design cannot be developed with one of these pile sections at the minimum spacing, another pier type should be selected. The outside piles shall be battered 2" per foot, and the inside piles shall be driven vertically. WisDOT does not rely on the shell of CIP piles for capacity; therefore these piles are less of a concern for long term reduced capacity due to corrosion than steel H-piles. For that reason the BOS Development Chief must give approval for the use of steel H-piles in open pile bents.

Because of the minimum pile spacing, the superstructure type used with pile bents is generally limited to cast-in-place concrete slabs, prestressed girders and steel girders with spans under approx. 70' and precast, prestressed box girders less than 21" in height.

To ensure that pile bents are capable of resisting the lateral forces resulting from floating ice and debris or expanding ice, the maximum distance from the top of the pier cap to the stable streambed elevation, including scour, is limited to:

- 15' for 12¾" diameter piles (or 12" H-piles if exception is granted).
- 20' for 14" diameter piles (or 14" H-piles if exception is granted).

Use of the pile values in Table 11.3-5 or Standard for Pile Details is valid for open pile bents due to the exposed portion of the pile being inspectable.

The minimum longitudinal reinforcing steel in cast-in-place piles with steel shells is 6-#7 bars in 12" piles and 8-#7 bars in 14" piles. The piles are designed as columns fixed from rotation in the plane of the pier at the top and at some point below streambed.

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

Pile bents shall meet the following criteria:

- If the water velocity, Q_{100}, is greater than 7 ft/sec, the quantity of the 100-year flood shall be less than 12,000 ft^3/sec.
- If the streambed consists of unstable material, the velocity of the 100-year flood shall not exceed 9 ft/sec.

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.

In most cases, pile encasement concrete may be poured underwater. While there are known risks with underwater concreting, this allowance has provided a cost-effective solution for small to intermediate stream crossings and past experience indicates concrete can be properly placed for encasement purposes. To help ensure minimum construction practices are being specified on a project, three pile encased pier types should be considered during the design process. These types are based on water depth and provide bid items, as necessary, to better ensure concrete can be properly placed. For the below discussions, water depth "H" is defined as the normal (or observed) water elevation minus the bottom of pier concrete elevation. Other factors such as velocity and the 2-year water elevation should also be considered when selecting pier types. The pile encased pier types are as follows:

- Type 1 (H ≤ 5.0'): For low water depths, the contractor may elect to furnish a cofferdam or other means to construct the pier per the plans and specifications. This may include underwater concreting or placing concrete in the dry. Additional bid items are not needed, and all work associated with properly constructing the pier shall be considered incidental to pier construction. Note: A cofferdam may be required due to environmental concerns. See 13.11.5 for additional guidance.

- Type 2 (5.0’ < H ≤ 10.0’): For moderate water depths, a cofferdam should be used to ensure that the concrete placed underwater is sound and to the limits shown on the plans. At a minimum, the cofferdam will remove the running water condition, stabilize excavations for the placement of forms, improve inspection conditions, and may allow dewatering, if needed. Bid item “Cofferdams (Structure)” is required and bid item “Underwater Substructure Inspection (Structure)” is required to inspect the concrete quality prior to removing the cofferdam.

- Type 3 (H > 10.0’): For high water depths, underwater concreting becomes increasingly difficult and is likely beyond the maximum practical depth for setting formwork and placing the reinforcing steel. As such, underwater concreting should be avoided and pier concrete should be placed in the dry. While this pier construction type may increase the initial pier construction cost, it will provide better quality concrete and avoid costly repairs. Alternative pier types (Hammerhead or Solid Wall) should also be considered during the design process to determine the most effective pier type. Bid item “Cofferdams (Structure)” is required and bid item “Concrete Masonry Seal” will most likely be required. The bid item “Underwater Substructure Inspection (Structure)” is not required when pier concrete will be poured in the dry.

Pile encased pier Types are detailed on Standard for Pile Encased Pier (Type). See 13.11.5 for additional guidance regarding cofferdams and seals. 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.

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 Chapter 4 for additional information about aesthetics.

13.3 Location

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

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

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

13.4 Loads on Piers

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

13.4.1 Dead Loads

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

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

13.4.2 Live Loads

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

WisDOT policy items:

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

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

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

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

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

13.4.3 Vehicular Braking Force

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

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

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

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

13.4.4 Wind Loads

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

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

The wind pressure (P_Z) shall be determined as:

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

Where:

P_Z = design wind pressure (ksf)

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

K_Z = pressure exposure and elevation coefficient

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

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

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

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

G = gust effect factor

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

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

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

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

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

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

Wind loads are divided into the following four types.

13.4.4.1 Wind Load from the Superstructure

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

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

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

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

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

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

Both forces shall be applied simultaneously.

WisDOT policy item:

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

Strength III:

- 0.044 ksf, transverse
- 0.011 ksf, longitudinal

Strength V:

- 0.021 ksf, transverse
- 0.006 ksf, longitudinal

Service I:

- 0.016 ksf, transverse
- 0.004 ksf, longitudinal

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

13.4.4.2 Wind Load Applied Directly to Substructure

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

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

WisDOT policy item:

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

Strength III:

- 0.054 ksf, transverse
- 0.054 ksf, longitudinal

Strength V:

- 0.026 ksf, transverse
- 0.026 ksf, longitudinal

Service I:

- 0.020 ksf, transverse
- 0.020 ksf, longitudinal

Both forces shall be applied simultaneously.

13.4.4.3 Wind Load on Vehicles

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

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

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

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

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

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

Both forces shall be applied simultaneously.

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

13.4.4.4 Vertical Wind Load

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

- 0.020 ksf (Strength III)

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

WisDOT policy item:

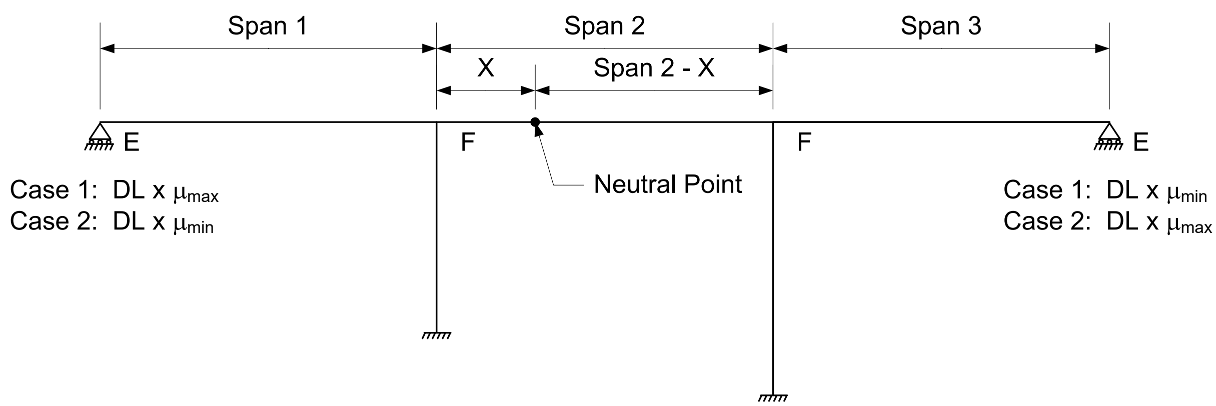
If WisDOT policy items are being applied in 13.4.4.1 and 13.4.4.2, assume the wind direction is perpendicular to the longitudinal axis of the bridge and apply the vertical wind load as described above.

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

13.4.5 Uniform Temperature Forces

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

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



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

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

Figure 13.4-1
Neutral Point Location with Multiple Fixed Piers

As used in Figure 13.4-1:

E = Column or shaft modulus of elasticity (ksi)

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

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

T = Temperature change of superstructure (°F)

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

h = Column height; top of footing to top of cap (ft)

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

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

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

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

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

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

Where:

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

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

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

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

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

Table 13.4-1
Temperature Expansion Values

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

13.4.6 Force of Stream Current

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

13.4.6.1 Longitudinal Force

The longitudinal force is computed as follows:

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

Where:

p = Pressure of flowing water (ksf)

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

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

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

13.4.6.2 Lateral Force

The lateral force is computed as follows:

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

Where:

p = Lateral pressure of flowing water (ksf)

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

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

Table 13.4-2
Lateral Drag Coefficient Values

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

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

13.4.7 Buoyancy

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

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

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

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

Table 13.4-3
Submerged Unit Weights of Various Soils

13.4.8 Ice

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

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

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

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

13.4.8.1 Force of Floating Ice and Drift

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

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

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

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

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

Where:

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

WisDOT policy item:

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

WisDOT policy item:

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

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

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

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

WisDOT exception to AASHTO:

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

13.4.8.2 Force Exerted by Expanding Ice Sheet

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

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

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

13.4.9 Centrifugal Force

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

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

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

Where:

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

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

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

13.4.10 Extreme Event Collision Loads

WisDOT policy item:

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

WisDOT policy item:

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

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

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

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

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

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

WisDOT exception to AASHTO:

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

13.5 Load Application

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

13.5.1 Loading Combinations

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

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

Table 13.5-1
Load Factors

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

13.5.2 Expansion Piers

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

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

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

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

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

Where:

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

Example E27-1.8 illustrates the calculation of this force.

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

13.5.3 Fixed Piers

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

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

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

13.6 Multi-Column Pier and Cap Design

WisDOT policy item:

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

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

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

WisDOT policy item:

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

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

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

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

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

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

13.7 Hammerhead Pier Cap Design

WisDOT policy item:

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

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

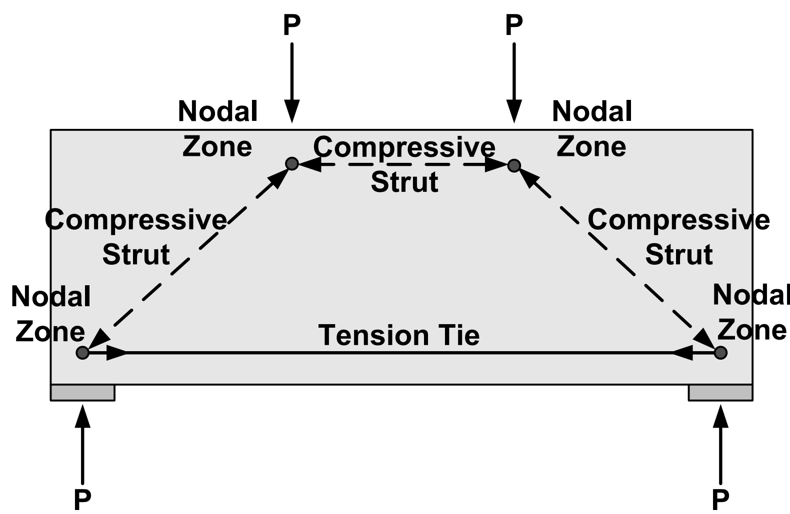


Figure 13.7-1
Basic Strut-and-Tie Elements

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

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

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

13.7.1 Draw the Idealized Truss Model

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

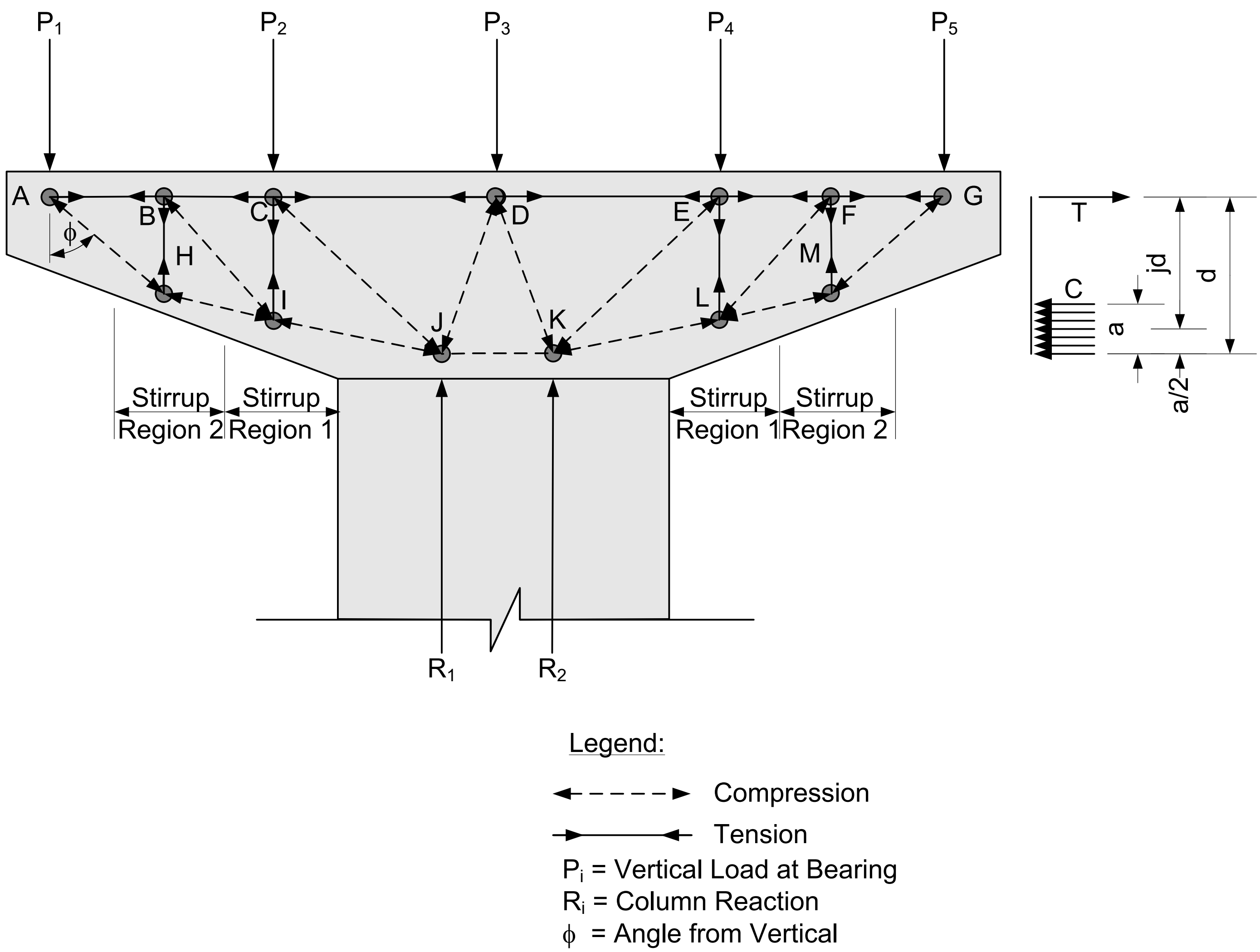


Figure 13.7-2
Example Hammerhead Pier Cap Strut-and-Tie Model

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

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

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

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

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

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

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

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

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

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

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

- CCC: Nodes where only struts intersect

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

- CTT: Nodes where ties intersect in two different directions

13.7.2 Solve for the Member Forces

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

13.7.3 Check the Size of the Bearings

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

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

Where:

ϕ	=	Resistance factor for bearing on concrete, equal to 0.70, as specified in **LRFD [5.5.4.2]**
P_u	=	Bearing reaction from strength limit state (kips)
P_n	=	Nominal bearing resistance (kips)

The nominal bearing resistance of the node face shall be:

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

Where:

f_{cu}	=	Limiting compressive stress at the face of a node **LRFD [5.8.2.5.3]** (ksi)
A_{cn}	=	Effective cross-sectional area of the node face **LRFD [5.8.2.5.2]** (in^2)

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

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

Where:

f'_c	=	Compressive strength of concrete (ksi)
m	=	Confinement modification factor **LRFD [5.6.5]**

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

For node regions with bearings:

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

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

- Node regions with no crack control reinforcement:

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

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

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

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

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

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

13.7.4 Design Tension Tie Reinforcement

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

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

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

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

Where:

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

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

f_y = Yield strength of reinforcement (ksi)

P_n = Nominal resistance of tension tie (kips)

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

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

Vertical tension ties, such as ties B-H and C-I, are used to determine the vertical stirrup requirements in the cap. Similar to traditional shear design, two stirrup legs shall be accounted for when computing A_{st}. In Figure 13.7-2, the number of stirrups, n, necessary to provide the A_{st} required for tie B-H shall be spread out across Stirrup Region 2. The length limit (L_2) of Stirrup Region 2 is from the midpoint between nodes A and B to the midpoint between nodes B and C. When vertical ties are located adjacent to columns, such as with tie C-I, the stirrup region extends to the column face. Therefore, the length limit (L_1) of Stirrup Region 1 is from the column face to the midpoint between nodes B and C. Using the equations above, the minimum area of reinforcement (A_{st}) can be found for the vertical tension tie **LRFD [5.8.2.4.1]**. The number of vertical stirrup legs at a cross-section can be selected, and their total area can be calculated as ($A_{stirrup}$). The number of stirrups required will then be:

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

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

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

Where:

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

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

13.7.5 Check the Compression Strut Capacity

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

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

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

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

Where:

P_n = Nominal resistance of compression strut (kips)

P_u = Compression strut force from strength limit state (kips)

ϕ = Resistance factor for compression in strut-and-tie models, equal to 0.70, as specified in **LRFD [5.5.4.2]**

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

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

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

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

Where:

f'_c = Compressive strength of concrete (ksi)

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

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

For node regions with struts:

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

- Node regions with no crack control reinforcement:

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

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

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

$P_u \leq \phi \cdot (0.85 \cdot f'_c) \cdot A_{cn}$ --- (back face) --- (CCC) Node

$P_u \leq \phi \cdot (0.70 \cdot f'_c) \cdot A_{cn}$ --- (back face) --- (CCT) Node

$P_u \leq \phi \cdot (0.65 \cdot f'_c) \cdot A_{cn}$ --- (back face) --- (CTT) Node

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

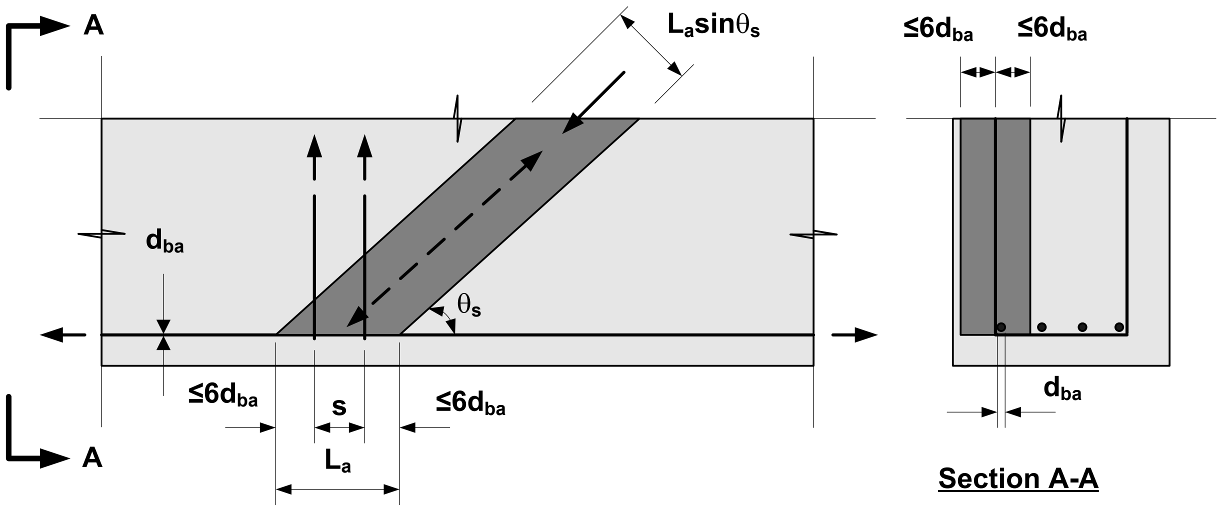


Figure 13.7-3
Strut Anchored by Tension Reinforcement Only (CTT)

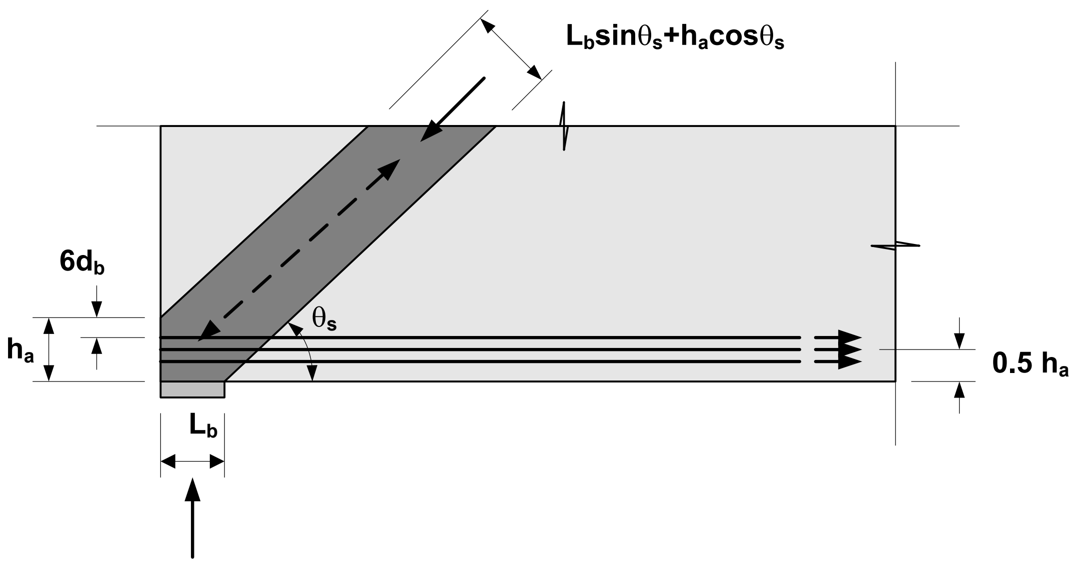


Figure 13.7-4
Strut Anchored by Bearing and Tension Reinforcement (CCT)

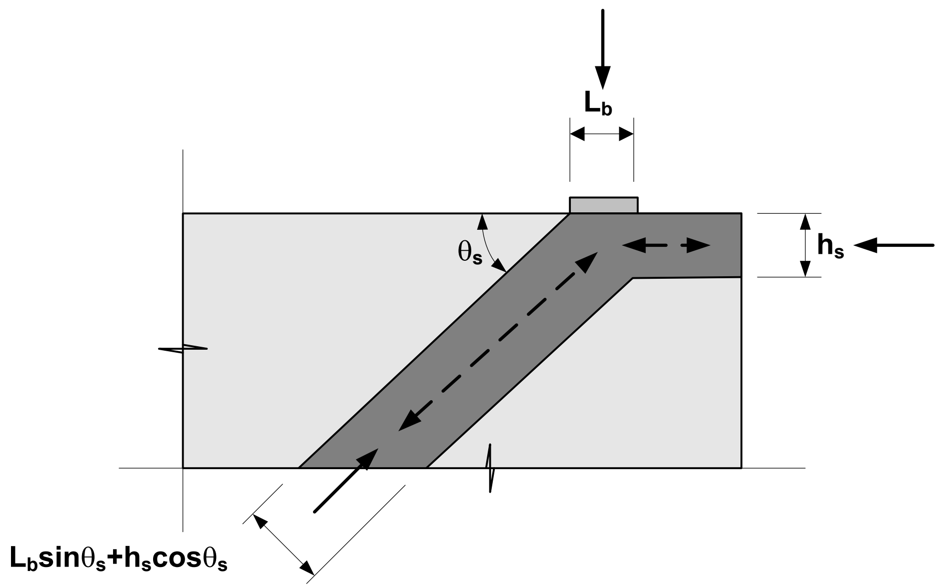


Figure 13.7-5
Strut Anchored by Bearing and Strut (CCC)

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

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

13.7.6 Check the Tension Tie Anchorage

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

13.7.7 Provide Crack Control Reinforcement

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

The reinforcement in the vertical direction shall satisfy:

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

The reinforcement in the horizontal direction shall satisfy:

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

Where:

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

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

b_w = Width of member (in.)

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

13.8 General Pier Cap Information

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

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

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

Where:

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

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

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

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

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

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

13.9 Column / Shaft Design

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

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

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

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

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

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

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

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

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

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

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

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

13.10 Pile Bent and Pile Encased Pier Analysis

WisDOT policy item:

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

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

WisDOT policy item:

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

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

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

Where:

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

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

In which:

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

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

13.11 Footing Design

13.11.1 General Footing Considerations

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

For multi-columned piers:

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

For hammerhead piers:

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

WisDOT exception to AASHTO:

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

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

13.11.2 Isolated Spread Footings

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

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

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

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

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

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

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

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

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

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

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

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

    a. Two-way action

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

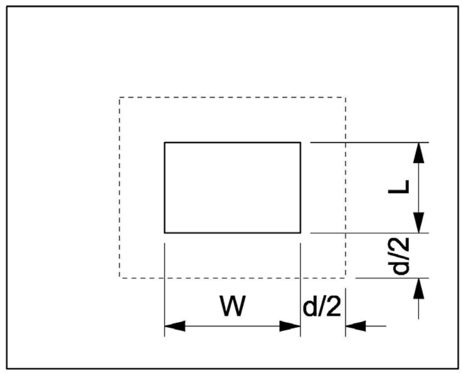


Figure 13.11-1
Critical Perimeter Location for Spread Footings

b. One-way action

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

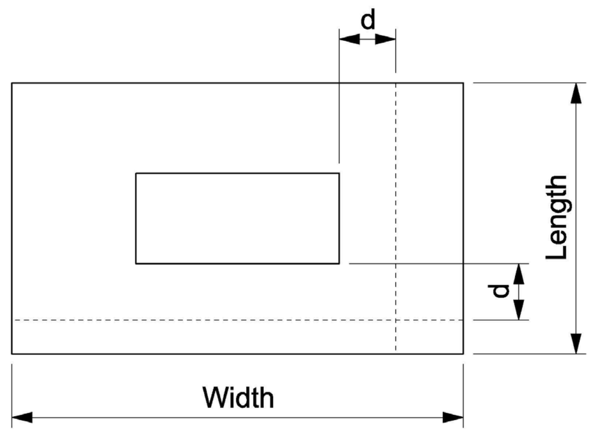


Figure 13.11-2
Shear Location for One-Way Action

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

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

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

13.11.3 Isolated Pile Footings

WisDOT policy item:

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

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

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

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

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

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

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

3. Same as spread footing.

4. Same as spread footing.

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

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

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

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

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

   a. Two-way action

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

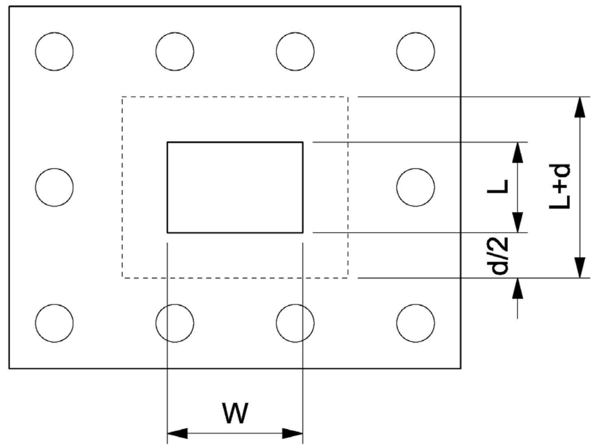


Figure 13.11-3
Critical Perimeter Location for Pile Footings

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

b. One-way action

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

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

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

13.11.4 Continuous Footings

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

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

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

13.11.5 Cofferdams and Seals

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

A cofferdam bid item may be warranted when water is expected at a concrete substructure unit during construction. The cofferdam shall be practically watertight to allow for dewatering such that the substructure is constructed in a dry environment. An exception is for pile encased piers. These substructures can be poured underwater, but in certain cases may still require a cofferdam for protection and/or to address environmental concerns. The designer should consult with geotechnical and regional personnel and the pile encased pier guidance provided in 13.2.3 to determine if a cofferdam is required. If a cofferdam is warranted, then include the bid item “Cofferdams (Structure)”.

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

A seal is a mat of unreinforced concrete poured under water inside a cofferdam. The seal is designed to withstand the hydrostatic pressure on its bottom when the water above it is removed. For shallow water depths and certain soil conditions a concrete seal may not be necessary in order to dewater a cofferdam. Coordinate with geotechnical personnel to determine if a concrete seal is required. The designer shall determine if a concrete seal is required for a cofferdam. For pile encased piers, see guidance provided in 13.2.3 to determine if a seal is required. If a concrete seal is required, then include the bid item “Concrete Masonry Seal” and required seal dimensions. The cofferdam design shall be the responsibility of the contractor.

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

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

Table 13.11-1
Bond on Piles and Sheet Piling

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

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

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

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

Assume 15' x 18' x 3.25' seal.

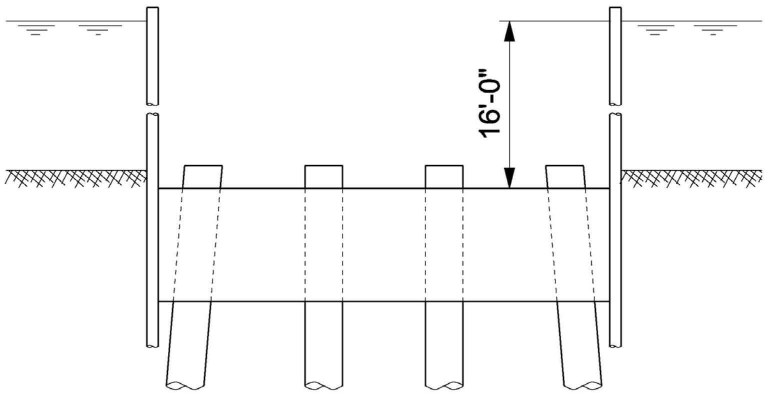


Figure 13.11-4
Seal Inside a Cofferdam

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

332 > 324 OK

USE 3'- 3" THICK SEAL

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

13.12 Quantities

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

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

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

13.13 Design Examples

This page intentionally left blank.

Table of Contents

and/or bicycle traffic, a live load of 90 psf is used. Consideration should also be given to maintenance vehicle loads as specified in Chapter 37 – Pedestrian Bridges.

17.2.5 Load Factors

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

For the design limit states, the values of γ_i for different types of loads are found in **LRFD [Tables 3.4.1-1 and 3.4.1-2]**. Load factors most commonly used for superstructure design are also presented in Table 17.2-5.

Load Combination	Load Factor, γi				
	DC		DW		LL+IM
	Maximum	Minimum	Maximum	Minimum	
Strength I	1.25	0.90	1.50	0.65	1.75
Strength III	1.25	0.90	1.50	0.65	0.00
Strength V	1.25	0.90	1.50	0.65	1.35
Service I	1.00	1.00	1.00	1.00	1.00
Service II	1.00	1.00	1.00	1.00	1.30
Service III	1.00	1.00	1.00	1.00	0.80
Fatigue I	0.00	0.00	0.00	0.00	1.75
Extreme Event II	1.25	0.90	1.50	0.65	0.50

Table 17.2-5
Load Factors

The maximum and minimum values should be used to maximize the intended effect of the load. An example of the use of minimum load factors is the load factor for dead load when uplift is being checked.

17.2.6 Resistance Factors

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

Resistance factors are presented in **LRFD [1.3.2.1]**, **LRFD [5.5.4.2]**, **LRFD [6.5.4.2]**, **LRFD [6.5.5]** and **LRFD [6.10.1.7]**. The most commonly used resistance factors for superstructure design are also presented in Table 17.2-6.

Limit State	Material	Application	Resistance Factor, ϕ
Strength	Concrete	Flexure (reinforced concrete)	0.90
		Flexure (prestressed concrete)	1.00
		Shear (normal weight)	0.90
		Shear (lightweight)	0.90
	Steel	Flexure	1.00
		Shear	1.00
		Axial compression, steel only	0.90
		Axial compression, composite	0.90
		Tension, fracture in net section	0.80
		Tension, yielding in gross section	0.95
		Bolts bearing on material	0.80
		Shear connectors	0.85
		A325 and A490 bolts in tension	0.80
		A325 and A490 bolts in shear	0.80
		A307 bolts in tension	0.80
		A307 bolts in shear	0.65
		Block shear	0.80
		Web crippling	0.80
		Welds	See **LRFD [6.5.4.2]**
Service	All	All	1.0
Fatigue	All	All	1.0
Extreme Event	All	All	1.0

Table 17.2-6
Resistance Factors

17.2.7 Distribution of Loads for Slab Structures

For slab structures, the distribution of loads is based on strip widths, as illustrated in Figure 17.2-6 through Figure 17.2-11. Figure 17.2-6 and Figure 17.2-7 illustrate the distribution of loads for slab structures with no sidewalks. Figure 17.2-8 and Figure 17.2-9 illustrate the distribution of loads for slab structures with sidewalks. Figure 17.2-10 and Figure 17.2-11 illustrate the distribution of loads for slab structures with raised sidewalks. It should be noted that, although medians are not shown in these figures, medians are treated similar to other superimposed dead loads.

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”.

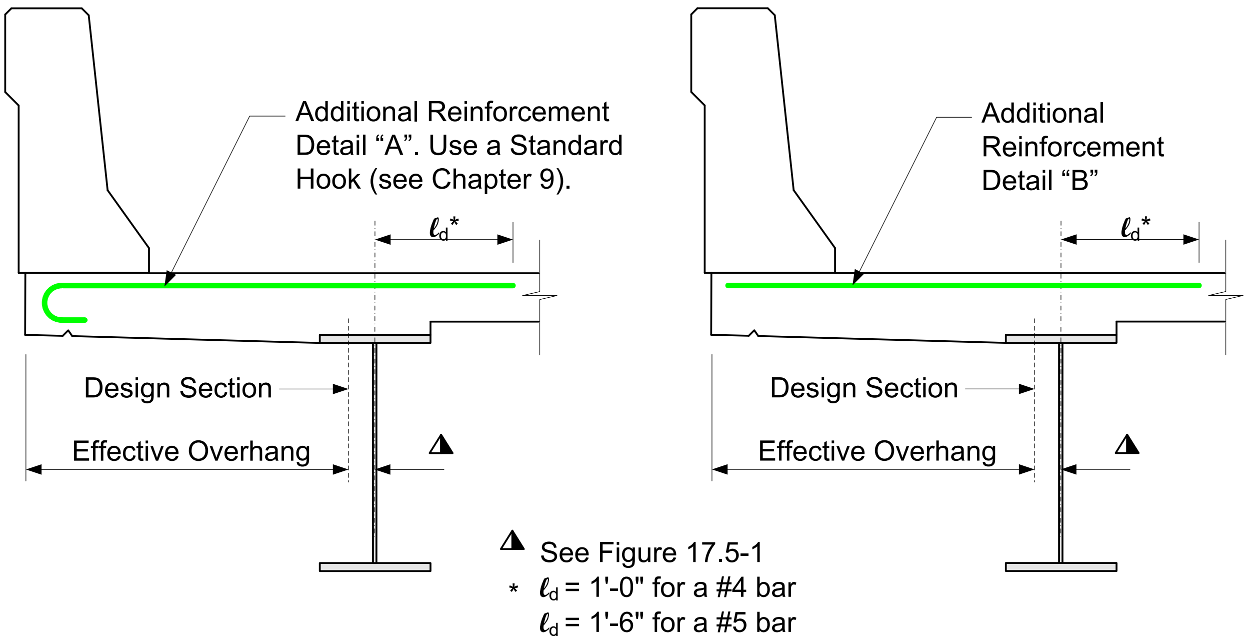


Figure 17.6-8
Overhang Reinforcement Details

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 120 feet or the width of a slab superstructure exceeds 52 feet, a longitudinal construction joint with reinforcement through the joint shall be detailed. For decks between 90 and 120 feet, an optional joint shall be detailed. Longitudinal joints should not be located directly above girders and should be at least 6 inches from the edge of the top flange of the girder. Longitudinal joints are 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. Longitudinal construction joint details are provided in Standard Details 24.11 – Slab Pouring Sequence and 18.02 – Continuous Flat Slab.

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.

Table of Contents

μ = Friction factor specified in **LRFD [5.7.4.4]**. This value shall be taken as 1.0 for WisDOT standard girders with a cast-in-place deck (dim.)

A_{vf} = Area of interface shear reinforcement crossing the shear plan within the area A_{cv} (in^2)

f_y = Yield stress of shear interface reinforcement not to exceed 60 (ksi)

P_c = Permanent net compressive force normal to the shear plane (kips)

P_c shall include the weight of the deck, haunch, parapets, and future wearing surface. A conservative assumption that may be considered is to set $P_c = 0.0$.

The nominal interface shear resistance, V_{ni}, shall not exceed the lesser of:

$V_{ni} \leq K_1 f'_c A_{cv}$ or $V_{ni} \leq K_2 A_{cv}$

Where:

K_1 = Fraction of concrete strength available to resist interface shear as specified in **LRFD [5.7.4.4]**. This value shall be taken as 0.3 for WisDOT standard girders with a cast-in-place deck (dim.)

K_2 = Limiting interface shear resistance as specified in **LRFD [5.7.4.4]**. This value shall be taken as 1.8 ksi for WisDOT standard girders with a cast-in-place deck

WisDOT policy item:

The stirrups that extend into the deck slab presented on the Standards are considered adequate to satisfy the minimum reinforcement requirements of **LRFD [5.7.4.2]**

19.3.3.16 Web Shear Reinforcement

Web shear reinforcement consists of placing conventional reinforcement perpendicular to the axis of the girder.

WisDOT policy item:

Web shear reinforcement shall be designed by **LRFD [5.7.3.4.2]** (General Procedure) using the Strength I limit state for WisDOT standard girders.

WisDOT prefers girders with spacing symmetrical about the midspan in order to simplify design and fabrication. The designer is encouraged to simplify the stirrup arrangement as much as possible. For vertical stirrups, the required area of web shear reinforcement is given by the following equation:

$$A_v \geq \frac{(V_n - V_c)s}{f_y d_v \cot\theta} \quad \text{(or } 0.0316\lambda\sqrt{f'_c}\frac{b_v s}{f_y} \text{ minimum , **LRFD [5.7.2.5]**)}$$

Where:

A_v	=	Area of transverse reinforcement within distance, s (in^2)
V_n	=	Nominal shear resistance (kips)
V_c	=	Nominal shear resistance of the concrete (kips)
s	=	Spacing of transverse reinforcement (in)
f_y	=	Specified minimum yield strength of transverse reinforcement (ksi)
d_v	=	Effective shear depth as determined in **LRFD [5.7.2.8]** (in)
θ	=	Angle of inclination of diagonal compressive stresses as determined in **LRFD 5.7.3.4** (degrees)
b_v	=	Minimum web width within the depth d_v, (in)
λ	=	Concrete density modification factor; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

θ shall be taken as follows:

$$\theta = 29 + 3500\varepsilon_s$$

Where:

ε_s = Net longitudinal tensile strain in the section at the centroid of the tension reinforcement.

$$= \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps}f_{po}\right)}{E_s A_s + E_p A_{ps}}$$

Where:

$\lvert M_u \rvert$	=	Absolute value of the factored moment at the section, not taken less than $\lvert V_u - V_p \rvert d_v$ (kip-in.)
N_u	=	Factored axial force, taken as positive if tensile and negative if compression (kip)
V_p	=	Component of prestressing force in the direction of the shear force; positive if resisting the applied shear (kip)
A_{ps}	=	Area of prestressing steel on the flexural tension side of the member (in^2).
A_s	=	Area of nonprestressing steel on the flexural tension side of the member (in^2).

f_{po} = A parameter taken as modulus of elasticity of prestressing steel multiplied by the locked-in difference in strain between the prestressing steel and the surrounding concrete (ksi).

$$V_u = 1.25DC + 1.5DW + 1.75(LL + IM)$$
$$V_n = V_u / \phi$$

Where:

V_u = Strength I Limit State shear force (kips)

ϕ = 0.90 per **LRFD [5.5.4.2]**

See 17.2 for further iformation regarding load combinations.

Per **LRFD [5.7.3.3]**, determine V_c as given by:

V_c = $0.0316\beta\lambda\sqrt{f'_c}\, b_v d_v$

Where:

β = Factor indicating ability of diagonally cracked concrete to transmit tension and shear. **LRFD [5.7.3.4]**

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

Where:

$\beta = \frac{4.8}{(1+750\varepsilon_s)}$ (For sections containing at least the minimum amount of transverse reinforcement specified in **LRFD [5.7.2.5]**)

WisDOT policy item:

Based on past performance, for prestressed I-girders the upper limit for web reinforcement spacing, s_{max}, per **LRFD [5.7.2.6]** will be reduced to 18 inches.

When determining shear reinforcement, spacing requirements as determined by analysis at 1/10th points, for example, should be carried-out to the next 1/10th point. As an illustration, spacing requirements for the 1/10th point should be carried out to very close to the 2/10th point, as the engineer, without a more refined analysis, does not know what the spacing requirements would be at the 0.19 point. For the relatively small price of stirrups, don't shortchange the shear capacity of the prestressed girder.

The web reinforcement spacing shall not exceed the maximum permitted spacing determined as:

- If $\upsilon_u < 0.125f'_c$, then s_{max} = $0.8d_v \leq 18"$
- If $\upsilon_u \geq 0.125f'_c$, then s_{max} = $0.4d_v \leq 12"$

Where:

$$\upsilon_u = \frac{V_u - \phi V_p}{\phi b_v d_v}$$ per **LRFD [5.7.2.8]**.

The nominal shear resistance, $V_c + V_s$, is limited by the following:

$$V_c + \frac{A_v f_y d_v \cot\theta}{s} \leq 0.25f'_c b_v d_v$$

Reinforcement in the form of vertical stirrups is required at the extreme ends of the girder. The stirrups are designed to resist 4% of the total prestressing force at transfer at a unit stress of 20 ksi and are placed within h/4 of the girder end, where h is the total girder depth. For a distance of 1.5d from the ends of the beams, reinforcement shall be placed to confine the prestressing steel in the bottom flange. The reinforcement shall be shaped to enclose the strands, shall be a #3 bar or greater and shall be spaced at less than or equal to 6”. Note that the reinforcement shown on the Standard Details sheets satisfies these requirements.

Welded wire fabric may be used for the vertical reinforcement. It must be deformed wire with a minimum size of D18.

Per **LRFD [5.7.3.5]**, at the inside edge of the bearing area to the section of critical shear, the longitudinal reinforcement on the flexural tension side of the member shall satisfy:

$$A_s f_y + A_{ps} f_{ps} \geq \left(\frac{V_u}{\phi} - 0.5V_s \right) \cot\theta$$

In the above equation, $\cot\theta$ is as defined in the V_c discussion above, and V_s is the shear reinforcement resistance at the section considered. Any lack of full reinforcement development shall be accounted for. Note that the reinforcement shown on the Standard Detail sheets satisfies these requirements.

19.3.3.17 Continuity Reinforcement

The design of non-prestressed reinforcement for negative moment at the support is based on the Strength I limit state requirements of **LRFD [5.6.3]**:

$$M_u = 1.25DC + 1.50DW + 1.75(LL + IM)$$

28" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	59	65
6’-6”	58	63
7’-0”	56	62
7’-6”	55	60
8’-0”	54	59
8’-6”	52	57
9’-0”	51	56
9’-6”	50	54
10’-0”	49	53
10’-6”	48	52
11’-0”	47	51
11’-6”	46	50
12’-0”	45	48

36" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	76	82
6’-6”	74	80
7’-0”	69	78
7’-6”	66	76
8’-0”	65	75
8’-6”	63	69
9’-0”	62	67
9’-6”	60	65
10’-0”	59	64
10’-6”	58	63
11’-0”	51	61
11’-6”	50	60
12’-0”	49	58

36W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	94	101
6’-6”	92	99
7’-0”	90	97
7’-6”	88	95
8’-0”	87	93
8’-6”	85	91
9’-0”	83	90
9’-6”	82	87
10’-0”	80	86
10’-6”	79	84
11’-0”	77	82
11’-6”	76	81
12’-0”	73	79

45W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	111	120
6’-6”	109	117
7’-0”	107	115
7’-6”	105	113
8’-0”	103	111
8’-6”	101	108
9’-0”	99	106
9’-6”	97	104
10’-0”	95	102
10’-6”	94	100
11’-0”	92	98
11’-6”	90	96
12’-0”	88	94

Table 19.3-1
Maximum Span Length vs. Girder Spacing

54W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	125	134
6'-6"	123	132
7'-0"	120	129
7'-6"	118	127
8'-0"	116	125
8'-6"	114	122
9'-0"	112	120
9'-6"	110	117
10'-0"	108	115
10'-6"	106	114
11'-0"	104	111
11'-6"	103	110
12'-0"	100	107

72W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	153*	164*⊗
6'-6"	150	161*⊗
7'-0"	148	158*
7'-6"	145	156*
8'-0"	143	153*
8'-6"	140	150
9'-0"	138	148
9'-6"	135	144
10'-0"	133	142
10'-6"	131	140
11'-0"	129	137
11'-6"	127	135
12'-0"	124	132

82W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	166*⊗	177*⊗
6'-6"	163*⊗	174*⊗
7'-0"	161*⊗	172*⊗
7'-6"	158*	169*⊗
8'-0"	156*	166*⊗
8'-6"	152	163*⊗
9'-0"	150	160*⊗
9'-6"	147	157*
10'-0"	145	154*
10'-6"	143	152
11'-0"	140	149
11'-6"	138	147
12'-0"	135	144

Table 19.3-2
Maximum Span Length vs. Girder Spacing

* For lateral stability during lifting these girder lengths may require pick-up point locations greater than distance d (girder depth) from the ends of the girder. The designer shall assume that the pick-up points will be at the 1/10 points from the end of the girder and provide extra non-prestressed steel in the top flange, if required, and check the concrete strength near the

Table of Contents

23.4.12 Thermal Expansion

Thermal expansion may be neglected in spike "laminated decks". Generally, thermal expansion has not presented problems in wood deck systems. Most wood decks inherently contain gaps at the butt joints that can absorb thermal movements **LRFD [9.9.3.4]**.

23.4.13 Wearing Surfaces

Laminated decks shall be provided with a wearing surface conforming to the provisions of **LRFD [9.9.8]**. Experience has shown that unprotected wood deck surfaces are vulnerable to wear and abrasion and/or may become slippery when wet.

23.4.14 Deck Tie-Downs

Where deck panels are attached to wood supports, the tie-downs shall consist of metal brackets that are bolted through the deck and attached to the sides of the supporting component. Lag screws or deformed shank spikes may be used to tie panels down to the wood support **LRFD [9.9.4.2]**.

23.4.15 Transverse Stiffener Beam

Interconnection of panels should be made with transverse stiffener beams attached to the underside of the deck. The distance between stiffener beams shall not exceed 8 feet, and the rigidity, EI, of each stiffener beam shall not be less than 80,000 kip-in^2. The beams shall be attached to each deck panel near the panel edges and at intervals not exceeding 15 inches **LRFD [9.9.4.3.1]**.

23.4.16 Metal Fasteners and Hardware

Attachments and fasteners used in wood construction shall be of stainless steel , malleable iron, aluminum or steel that is galvanized, cadmium plated, or otherwise coated to provide durability **LRFD [2.5.2.1.1]**. Material property requirements for metal fasteners and hardware are covered in **LRFD [8.4.2]**. The design of fasteners and connections is covered in **LRFD [8.13]**.

23.4.17 Preservative Treatment

All wood used for permanent applications shall be pressure impregnated with wood preservatives in accordance with the requirements of *AASHTO Standard Specifications for Transportation Materials* M133. Insofar as is practicable, all wood components shall be designed and detailed to be cut, drilled, and otherwise fabricated prior to pressure treatment with wood preservatives. When cutting, boring or other fabrication is necessary after preservative treatment, exposed, untreated wood shall be specified to be treated in accordance with the requirements of *AASHTO* M133. See **LRFD [8.4.3]** for other preservative treatment requirements.

23.4.18 Timber Rail System

Use approved crash-tested rail systems only.

23.4.19 Rating of Superstructure

Refer to *AASHTO Manual for Bridge Evaluation (MBE)* and also the example that follows.

Table of Contents

24.2 Materials

Structural steels currently used conform to ASTM A709 Specifications designated Grades 36, 50 and 50W. *AASHTO LRFD* gives the necessary design information for each grade of steel. Steel girders may utilize High-Performance Steel (HPS); however it may come at a premium price due to the limited number of mills that are rolling HPS. The limited number of mills may also have adverse effects on the delivery schedule.

HPS is currently produced by either quenching and tempering (Q&T) or by thermo-mechanical-controlled-processing (TMCP). TMCP HPS is currently available in plate thicknesses up to 2" and in maximum plate lengths from approximately 50' to 125' depending on weights. Q&T HPS is available in plate thicknesses from 2" to 4" (or less for larger plate widths), but because of the furnaces that are used in the tempering process, it is subject to a maximum plate-length limitation of 600" (50') or less, depending on weights. Therefore, whenever Q&T HPS is used (generally when HPS plates over 2" in thickness are specified), the maximum plate-length limitation should be considered when laying out flange (and web) transitions in a girder.

For fracture toughness, HPS provides significant toughness improvements given, that by default, Charpy V-notch requirements satisfy the more stringent Zone 3 requirements in all temperature zones. For welding, most of the bridge steels specified in the ASTM A709 Specifications can be welded without special precautions or procedures. However, special procedures should be followed to improve weldability and ensure high-quality welds when HPS is used.

Hybrid girder design utilizing HPS Grade 70 steel (Grade 70 is only available in HPS) for the flanges and Grade 50 steel for the web may be considered as a viable alternative. Such an arrangement has recently proven to be a popular option, primarily in regions of negative flexure.

For unpainted structures over stream crossings, Grade 50W weathering steel is recommended throughout.

Cracks have been observed in steel girders due to fabrication, fatigue, brittle fractures and stress corrosion. To insure against structural failure, the material is tested for plane-strain fracture toughness. As a result of past experience, the Charpy V-notch test is currently required on all grades of steel used for girders.

Plate width and length availability is an important consideration when it comes to sizing girder flanges. The availability of plate material varies from mill to mill. Generally, plates are available in minimum widths ranging from 48" to 60" and in maximum widths ranging from 150" to 190". *AASHTO/NSBA Steel Bridge Collaboration*, "*Guidelines to Design for Constructibility, G12.1*" (2016) contains some example plate length and width availability information from a single mill. However, a fabricator and/or mill should be consulted regarding the most up-to-date plate availability information. The maximum available plate length is generally a function of the plate width and thickness, steel grade and production process.

For additional information about plate widths and lengths, including maximum sizes for shipping and erection, see 24.4.6.2.

For additional information about materials, see Chapter 9 – Materials.

24.2.1 Bars and Plates

Bars and plates are grouped under flat rolled steel products that are designated by size as follows:

- Bars – 8" or less in width
- Plates – over 8" in width

<u>WisDOT policy item:</u>

AASHTO LRFD allows a minimum thickness of 5/16” for most structural steel members. Current WisDOT policy is to employ a minimum thickness of 7/16" for primary members and a minimum of 3/8" for secondary structural steel members.

Optional splices are permitted on plates which are detailed over 60' long. Refer to the latest steel product catalogs for steel sections and rolled stock availability.

24.2.2 Rolled Sections

A wide variety of structural steel shapes are produced by steel manufacturers. Design and detail information is available in the *AISC Manual of Steel Construction*, and information on previously rolled shapes is given in *AISC Iron and Steel Beams 1873 to 1952*. Refer to the latest steel product catalogs for availability and cost, as some shapes are not readily available and their use could cause costly construction delays.

24.2.3 Threaded Fasteners

The design of bolted connections is covered in **LRFD [6.13.2]**. As specified in **LRFD [6.13.2.1]**, bolted steel parts must fit solidly together after the bolts are tightened. The bolted parts may be coated or uncoated. It must be specified in the contract documents that all joint surfaces, including surfaces adjacent to the bolt head and nut, be free of scale (except for tight mill scale), dirt or other foreign material. All material within the grip of the bolt must be steel.

High-strength bolts are installed to have a specified initial tension, which results in an initial pre-compression between the joined parts. At service load levels, the transfer of the loads between the joined parts may then occur entirely via friction, with no bearing of the bolt shank against the side of the hole. Until the friction force is overcome, the shear resistance of the bolt and the bearing resistance of the bolt hole will not affect the ability to transfer the load across the shear plane between the joined parts.

In general, high-strength bolted connections designed according to *AASHTO LRFD* will have a higher reliability than the connected parts because the resistance factors for the design of bolted connections were selected to provide a higher level of reliability than those chosen for member design. Also, the controlling strength limit state in the connected part (for example, yielding or deflection) is typically reached well before the controlling strength limit state in the

$$b_{fc} \geq L/85$$

Where:

L = Length of the girder shipping piece

Satisfaction of this simple guideline can also help ensure that individual field sections will be stable for handling both in the fabrication shop and in the field. Adherence to this guideline can also facilitate erection without any required special stiffening trusses or falsework. It is recommended that the above two equations be used to establish a minimum required top-flange width in regions of positive flexure in composite girders.

As a practical matter, fabricators order flange material from wide plate, typically between 72” and 96” wide. They either weld the shop splices in the individual flanges after cutting them to width or they weld the different thickness plates together to form one wide plate and then strip the individual flanges. In the latter case, the individual flange widths must be kept constant within an individual shipping piece, which is preferred. Changing of flange widths at shop splices should be avoided if at all possible. Stripping the individual flanges from a single wide plate allows for fewer weld starts and stops and results in only one set of run-on and run-off tabs. It is estimated that up to 35% of the labor required to join the flanges can be saved by specifying changes in thickness rather than width within a field section.

A fabricator will generally order plate with additional width and length for cutting tolerance, sweep tolerance and waste. Waste is a particular concern when horizontally curved flanges are cut curved. The engineer should give some consideration as to how the material might be ordered and spliced; a fabricator can always be consulted for assistance. Flanges should be sized (including width, thickness and length) so that plates can be ordered and spliced with minimal waste. *AASHTO/NSBA Steel Bridge Collaboration*, “*Guidelines to Design for Constructability, G12.1*” (2016) is a free publication available from AASHTO which contains some specific recommendations and illustrative examples related to this issue.

The following additional guidelines are used for plate girder design and detailing:

1. Maximum change in flange plate thickness is 1” and preferably less.

2. The thinner plate is not less than 1/2 the thickness of the thicker flange plate.

3. Plate thicknesses are given in the following increments:

4. 1/16” up to 1”

5. 1/8” between 1” and 2”

6. 1/4” above 2”

7. Minimum plate size on the top flange of a composite section in the positive moment region is variable depending on the depth of web, but not less than 12” x ¾” for web

depths less than or equal to 66” and 14” x ¾” for web depths greater than 66”. Thinner plates become wavy and require extra labor costs to straighten within tolerances.

8. For plate girder flange widths, use 2” increments.

9. For plate girder web depths, use 3” increments.

10. Changes in plate widths or depths are to follow recommended standard transition distances and/or radii. The minimum size flange plates of 16" x 1 1/2" at the point of maximum negative moment and 16" x 1" for the bottom flange at the point of maximum positive moment are recommended for use on plate girders. The use of a minimum flange width on plate girders is necessary to maintain adequate stiffness in the girder so it can be fabricated, transported and erected. Deeper web plates with small flanges may use less steel, but they create problems during fabrication and construction. However, flange sizes on plate girders with web depths 48" or less may be smaller.

11. Flange plate sizes are detailed based on recommended maximum span lengths given in Table 24.4-1 for parallel flanged girders. The most economical girder is generally the one having the least total weight but is determined by comparing material costs and welding costs for added stiffener details. Plates over 60'-90’ (depending on thickness and material) are difficult to obtain, and butt splices are detailed to limit flange plates to these lengths or less. It is better to detail more flange butt splices than required and leave the decision to utilize them up to the fabricator. All butt splices are made optional to the extent of available lengths, and payment is based on the plate sizes shown on the plans. As previously described, detail flange plates to the same width and vary the thicknesses. This allows easier fabrication when cutting plate widths. Change widths, if necessary, only at field splices.

12. Minimum web thickness is 7/16" for girder depths less than or equal to 60". An economical web thickness usually has a few transverse stiffeners. Refer to 24.10 for transverse stiffener requirements. Due to fatigue problems, use of longitudinal stiffeners for plate girders is not encouraged.

24.4.7 Welding

Welding design details shall conform to current requirements of *Bridge Welding Code: AASHTO/AWS-D1.5*. Weld details are not shown on the plans but are specified by using standard symbols as given on Figure 24.4-2 and Figure 24.4-3. Weld sizes are based on the size required due to stress or the minimum size for plate thicknesses being connected.

24.6 Design Approach - Steps in Design

24.6.1 Obtain Design Criteria

The first design step for a steel girder is to choose the correct design criteria. The design criteria include the following:

- Number of spans
- Span lengths
- Skew angles
- Number of girders
- Girder spacing
- Deck overhang
- Cross-frame spacing
- Flange and web yield strengths
- Deck concrete strength
- Deck reinforcement strength
- Deck thickness
- Dead loads
- Roadway geometry
- Haunch depth

For steel girder design, the following load combinations are generally considered:

- Strength I
- Service II
- Fatigue I

The extreme event limit state (including earthquake load) is generally not considered for a steel girder design.

The following steps are taken in determining the girder or beam spacing and the slab thickness:

1. The girder spacing (and the resulting number of girders) for a structure is determined by considering the desirable girder depth and the span lengths. Refer to 24.4.2 for design aids. Where depth or deflection limitations do not control the design, it is usually more economical to use fewer girders with a wider spacing and a thicker slab. Four girders are generally considered to be the minimum, and five girders are desirable to facilitate future redecking.

2. The slab overhang on exterior girders is limited to 3'-7" measured from the girder centerline to the edge of slab. The overhang is limited to prevent rotation and bending of the web during construction caused by the forming brackets. The overhang width is generally determined such that the moments and shears in the exterior girder are similar to those in the interior girder. In addition, the overhang is set such that the positive and negative moments in the deck slab are balanced. A common rule of thumb is to make the overhang approximately 0.28 to 0.5 times the girder spacing. For girders less than, or equal to 36-inches in depth, limit the overhang to the girder depth, and preferably no wider than 0.80 the girder depth. The limits for raised sidewalk overhangs on the Standard for Median and Raised Sidewalk Details are likely excessive for such shallow girders.

3. Check if a thinner slab and the same number of members can be used by slightly reducing the spacing.

24.6.2 Select Trial Girder Section

Before the dead load effects can be computed, a trial girder section must be selected. This trial girder section is selected based on previous experience and based on preliminary design. Based on this trial girder section, section properties and dead load effects will be computed. Then specification checks will be performed to determine if the trial girder section successfully resists the applied loads. If the trial girder section does not pass all specification checks or if the girder optimization is not acceptable, then a new trial girder section must be selected and the design process must be repeated.

The following tips are presented to help bridge designers in developing an economical steel girder for most steel girder designs. Other design tips are available in various publications from the American Institute of Steel Construction (AISC) and from steel fabricators.

- Girder depth – The minimum girder depth is specified in **LRFD [2.5.2.6.3]**. An estimate of the optimum girder depth can be obtained from trial runs using design software. The web depth may be varied by several inches more or less than the optimum without significant cost penalty. Refer to 24.4.2 for recommended girder depths for a given girder spacing and span length.

- Web thickness – A "nominally stiffened" web (approximately 1/16 inch thinner than "unstiffened") will generally provide the least cost alternative or very close to it. However, for web depths of approximately 50” or less, unstiffened webs may be more economical.

- Plate transitions – For rolled sections, a change in section should occur only at field splice locations. For plate girders, include the change in section at butt splices and

24.16 Design Examples

This page intentionally left blank.

Table of Contents

27.1 General

Bridges supported in the conventional way by abutments and piers require bearings to transfer girder reactions without overstressing the supports, ensuring that the bridge functions as intended. Bridges usually require bearings that are more elaborate than those required for building columns, girders and trusses. Bridge bearings require greater consideration in minimizing forces caused by temperature change, friction and restraint against elastic deformations. A more detailed analysis in bridge bearing design considers the following:

- Bridges are usually supported by reinforced concrete substructure units, and the magnitude of the horizontal thrust determines the size of the substructure units. The coefficient of friction on bridge bearings should be as low as possible.

- Bridge bearings must be capable of withstanding and transferring dynamic forces and the resulting vibrations without causing eventual wear and destruction of the substructure units.

- Most bridges are exposed to the elements of nature. Bridge bearings are subjected to more frequent and greater total expansion and contraction movement due to changes in temperature than those required by buildings. Since bridge bearings are exposed to the weather, they are designed as maintenance-free as possible.

WisDOT policy item:

The temperature range considered for steel girder superstructures is -30°F to 120°F. A temperature setting table for steel bearings is used for steel girders; where 45°F is the neutral temperature, resulting in a range of 120° - 45° = 75° for bearing design.

The temperature range considered for prestressed concrete girder superstructures is 5°F to 85°F. Using an installation temperature of 60° for prestressed girders, the resulting range is 60° - 5° = 55° for bearing design. For prestressed girders, an additional shrinkage factor of 0.0003 ft/ft shall also be accounted for. (Do not include prestressed girder shrinkage when designing bearings for bridge rehabilitation projects). No temperature setting table is used for prestressed concrete girders.

See the Standard for Steel Expansion Bearing Details to determine bearing plate “A” sizing (steel girders) or anchor plate sizing (prestressed concrete girders). This standard also gives an example of a temperature setting table for steel bearings when used for steel girders.

WisDOT policy item:

According to **LRFD [14.4.1]**, the influence of dynamic load allowance need not be included for bearings. However, dynamic load allowance shall be included when designing bearings for bridges in Wisconsin. Apply dynamic load allowance in **LRFD [3.6.2]** to HL-93 live loads as stated in **LRFD [3.6.1.2, 3.6.1.3]** and distribute these loads, along with dead loads, to the bearings.

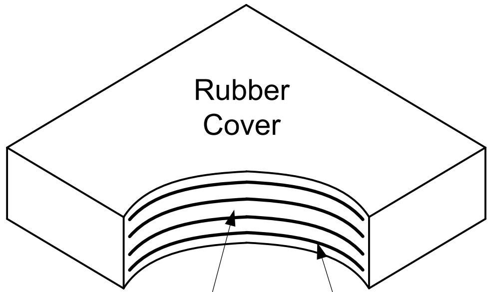


Figure 27.2-2
Laminated (Steel Reinforced) Elastomeric Bearing

AASHTO LRFD does not permit tapered elastomer layers in reinforced bearings. Laminated (steel reinforced) bearings must be placed on a level surface; otherwise gravity loads will produce shear strain in the bearing due to inclined forces. The angle between the alignment of the underside of the girder (due to the slope of the grade line, camber and dead load rotation) and a horizontal line must not exceed 0.01 radians, as per **LRFD [14.8.2]**. If the angle is greater than 0.01 radians or if the rotation multiplied by the top plate length is 1/8" or more, the 1 1/2" top steel plate must be tapered to provide a level load surface along the bottom of this plate under these conditions. The tapered plate will have a minimum thickness of 1 1/2" per *AASHTO LRFD Bridge Construction Specifications, Section 18*.

Plain and laminated (steel reinforced) elastomeric bearings can be designed by Method A as outlined in **LRFD [14.7.6]** and NCHRP-248 or by Method B as shown in **LRFD [14.7.5]** and NCHRP-298.

WisDOT policy item:

WisDOT uses Method A, as described in **LRFD [14.7.6]**, for elastomeric bearing design.

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, and WisDOT currently does not have a system in place to verify these requirements.

For several years, plain elastomeric bearing pads have performed well on prestressed concrete girder structures. Refer to the Standard for Bearing Pad Details for Prestressed Concrete Girders for details. Prestressed concrete girders using this detail are fixed into the concrete diaphragms at the supports, and the girders are set on 1/2" thick plain elastomeric bearing pads. Laminated (steel reinforced) bearing details and steel plate and elastomer thicknesses are given on the Standard for Elastomeric Bearings for Prestressed Concrete Girders.

The design of an elastomeric bearing generally involves the following steps:

1. Obtain required design input **LRFD [14.4 & 14.6]**

The required design input for the design of an elastomeric bearing at the service limit state is dead load, live load plus dynamic load allowance, minimum vertical force due to permanent load, and design translation. The required design input at the strength limit state is shear force. Other required design input is expansion length, girder or beam bottom flange width, minimum grade of elastomer, and temperature zone. Two temperature zones are shown for Wisconsin in **LRFD [Figure 14.7.5.2-1]**, zones C and D. WisDOT policy is for all elastomeric bearings to meet Zone D requirements.

2. Select a feasible bearing type – plain or laminated (steel reinforced)

3. Select preliminary bearing properties **LRFD [14.7.6.2]**

   The preliminary bearing properties can be obtained from **LRFD [14.7.6.2]** or from past experience. The preliminary bearing properties include elastomer cover thickness, elastomer internal layer thickness, elastomer hardness, elastomer shear modulus, elastomer creep deflection, pad length, pad width, number of steel reinforcement layers, steel reinforcement thickness, steel reinforcement yield strength and steel reinforcement constant-amplitude fatigue threshold. WisDOT uses the following properties:

   - Elastomer cover thickness = 1/4"
   - Elastomer internal layer thickness = 1/2"
   - Elastomer hardness: Durometer 60 +/- 5
   - Elastomer shear modulus (G): 0.1125 ksi < G < 0.165 ksi
   - Elastomer creep deflection @ 25 years divided by instantaneous deflection = 0.30
   - Steel reinforcement thickness = 1/8"
   - Steel reinforcement yield strength = 36 ksi or 50 ksi
   - Steel reinforcement constant-amplitude fatigue threshold = 24 ksi

   However, not all of these properties are needed for a plain elastomeric bearing design.

4. Check shear deformation **LRFD [14.7.6.3.4]**

   Shear deformation, Δ_S, is the sum of deformation from thermal effects, Δ_{ST}, as well as creep and shrinkage effects, $\Delta_{Scr/sh}$ ($\Delta_S = \Delta_{ST} + \Delta_{Scr/sh}$).

   $$\Delta_{ST} = (\text{Expansion length})(\Delta_T)(\alpha)$$

   Where:

   Δ_T = Change in temperature (see 27.1) (degrees)

Table of Contents

30.1 Crash-Tested Bridge Railings and FHWA Policy

Notice: All contracts with a letting date after December 31, 2019 must use bridge rails and transitions meeting the 2016 Edition of MASH criteria for new permanent installation and full replacement.

WisDOT policy item:

For all Interstate structures, the 42SS parapet shall be used. For all STH and USH structures with a posted speed >= 45 mph, the 42SS parapet shall be used.

The timeline for implementation of the above policy is:

- All contracts with a letting date after December 31, 2019.
  (This is an absolute, regardless of when the design was started.)

- All preliminary designs starting after October 1, 2017
  (Even if the let is anticipated to be prior to December 31, 2019.)

Contact BOS should the 42" height adversely affect sight distance, a minimum 0.5% grade for drainage cannot be achieved, or for other non-typical situations.

Crash test procedures for full-scale testing of guardrails were first published in 1962 in the *Highway Research Correlation Services Circular 482*. This was a one-page document that specified vehicle mass, impact speed, and approach angle for crash testing and was aimed at creating uniformity to traffic barrier research between several national research agencies.

In 1974, the National Cooperative Highway Research Program (NCHRP) published their final report based on NCHRP Project 22-2, which was initiated to address outstanding questions that were not covered in *Circular 482*. The final report, NCHRP Report 153 – *"Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances,"* was widely accepted following publication; however, it was recognized that periodic updating would be required.

NCHRP Project 22-2(4) was initiated in 1979 to address major changes to reflect current technologies of that time and NCHRP Report 230, *"Recommended Procedures for the Safety Performance Evaluation of Highway Safety Appurtenances,"* was published in 1980. This document became the primary reference for full-scale crash testing of highway safety appurtenances in the U.S. through 1993.

In 1986, the Federal Highway Administration (FHWA) issued a policy memorandum that stated highway bridges on the National Highway System (NHS) and the Interstate Highway System (IHS) must use crash-tested railings in order to receive federal funding.

The American Association of State Highway and Transportation Officials (AASHTO) recognized that the evolution of roadside safety concepts, technology, and practices necessitated an update to NCHRP Report 230 approximately 7 years after its adoption. NCHRP Report 350, *"Recommended Procedures for the Safety Performance Evaluation of Highway Features,"* represented a major update to the previously adopted report. The updates

30.3 General Design Details

1. Epoxy coated bars are required for all concrete parapets, curbs, medians, and sidewalks.

2. Adhesive anchored parapets are allowed at interior Traffic Railing locations only when the adjacent exterior parapet is a crash test approved Traffic Railing per 30.2 (i.e., cast-in-place anchors are used at exterior parapet location). See Standards for Parapet Footing and Lighting Detail for more information.

3. Sign structures, sign trusses, and monotubes shall be placed on top of railings to meet the working width and zone of intrusion dimensions noted in FDM 11-45-2.3.1.1 and 11-45-2.3.6.2.3 respectively.

4. 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.

5. Provide for expansion movement in tubular railings where expansion devices or concrete parapet deflection joints exist on the structure plan details. The tubular railing splice should be located over the joint and spaced evenly between railing posts. The tubular railing splice should be made continuous with a movable internal sleeve. If tubular railing is employed on conventional structures where expansion joints are likely to occur at the abutments only, the posts may be placed at equal spacing provided that no post is nearer than 2 feet from deflection joints in the parapet at the piers.

6. Refer to Standard for Vertical Face Parapet “A” – for detailing concrete parapet or sidewalk deflection joints. These joints are used based on previous experience with transverse deck cracking beneath the parapet joints.

7. 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. <u>Therefore, slip forming of bridge parapets shall not be allowed</u>.

8. For beam guard type “W” railing, locate the expansion splice at a post or on either side of the expansion joint.

9. Sidewalks - If there is a Traffic Railing between the roadway and an at grade sidewalk, and the roadway side of the Traffic Railing is more than 11’-0” from the exterior edge of deck, access must be provided to the at grade sidewalk for the snooper truck to inspect the underside of the bridge. The sidewalk width must be 10’-0” clear between barriers, including fence (i.e., use a straight fence without a bend). For protective screening, the total height of parapet and fence need not exceed 8’-0”. The boom extension on most snooper trucks does not exceed 11’-0” so provision must be made to get the truck closer to the edge.

10. Where Traffic Railing is utilized between the roadway and an at grade sidewalk, early coordination with the roadway designer should occur to provide adequate clearances

off of the structure to allow for proper safety hardware placement and sidewalk width. Additional clearance may be required in order to provide a crash cushion or other device to protect vehicles from the blunt end of the interior Traffic Railing off of the structure.

11. On shared-use bridges, fencing height and geometry shall be coordinated with the Region and the DNR (or other agencies) as applicable. Consideration shall be given to bridge use (i.e., multi-use/snowmobile may require vertical and horizontal clearances to allow grooming machine passage) and location (i.e., stream crossing vs. grade separation).

12. Per **LRFD [13.7.1.1]**, the use of raised sidewalks on structures shall be restricted to roadways with a design speed of 45 mph or less. The height of curbs for sidewalks is usually 6 inches. This height is more desirable than higher heights with regards to safety because it is less likely to vault vehicles. However, a raised curb is not considered part of the safety barrier system. On structure rehabilitations, the height of sidewalk may increase up to 8 inches to match the existing sidewalk height at the bridge approaches. Contact the Bureau of Structures Development Section if sidewalk heights in excess of 8 inches are desired. See Standard for Median and Raised Sidewalk Details for typical raised sidewalk detail information.

13. Pedestrian loads, as described in the AASHTO LRFD Bridge Design Specifications, shall be used to not only design the pedestrian railings on the structure, but shall also be used to design stairway railings that are adjacent to the structure and are part of the contract.

WisDOT policy item:

Noise walls are not allowed on WisDOT bridges.

Contact BOS for discussion on project specific exceptions to this policy. For example, a possible exception would be if a new bridge replaces an existing bridge that currently has a noise wall. Offset requirements of **LRFD [15.8.4, Case 4]** would need to be followed.

Table of Contents

32.1 General

The Regional Office shall determine the utilities that will be affected by the construction of any bridge structure at the earliest possible stage. It shall be their responsibility to deal with these utilities and to provide location plans or any other required sketches for their information. When the utility has to be accommodated on the structure, the Regional Office shall secure approval from the representative of the utility and the Bureau of Structures for the location and method of support.

Due consideration shall be given to the weight of the pipes, ducts, etc. in the design of the beams and diaphragms. To insure that the function, aesthetics, painting and inspection of stringers of a structure are maintained, the following applies to the installation of utilities on structures:

1. Permanent installations, which are to be carried on and parallel to the longitudinal axis of the structure, shall be placed out of sight, between the fascia beams and above the bottom flanges, on the underside of the structure.

2. Conglomeration of utilities in the same bay shall be avoided in order to facilitate maintenance painting and future inspection of girders in a practical manner.

3. In those instances where the proposed type of superstructure is not adaptable to carrying utilities in an out-of-sight location in the underside of the structure, an early determination must be made as to whether or not utilities are to be accommodated and, if so, the type of superstructure must be selected accordingly.

32.4 Pipeline Expansion Joints

Allowances must be made for changes in pipe length due to thermal expansion and alternate contraction. While couplings will take care of the normal amount of expansion and contraction in each length of pipe, expansion joints are required where no flexible joints are used in the pipeline or when the amount of concentrated movement at one point in excess of the amount that can be safely absorbed by the standard coupling.

An expansion joint should be located in the pipeline adjacent to every point where expansion means are provided in the superstructure.

Use couplings to accommodate the differential movement between the structure and the line itself, and to provide flexibility to accommodate vibrations of the structure. Each coupling can safely accommodate up to 3/8 inch longitudinal movement.

Proper alignment is important to insure free and concentric movement of the slip-type expansion joint. Alignment guides should be designed to allow free movement in only one direction along the axis of the pipe and to prevent any horizontal or vertical movement of the pipe. Suitable pipe alignment guides may be obtained from reliable pipe alignment guide manufacturers. Alignment guides should be fastened to some rigid part of the installation, such as the framework of the bridge. Alignment guides should be located as close to the expansion joint as possible, up to a maximum of 4 pipe diameters. The distance from the first pipe guide to the second should not exceed a maximum of 14 pipe diameters from the first guide. Where an anchor is located adjacent to an expansion joint, it too, should be located as close to the expansion joint as possible – to a maximum of 4 pipe diameters from the expansion-joint. Additional pipe supports are usually required. Pipe supports should not be substituted for alignment guides.

The main pipe anchors must be designed to withstand the full thrust resulting from internal line pressure; also, the force required to collapse the slip pipe, and the frictional forces due to guides and supports.

32.5 Lighting

When lighting conduits are used in a bridge use an approved expansion fitting at each semi expansion or expansion joint.

Use bolted option on all bridges with X-frame and lower laterals. Do not use bolted option when channel diaphragms are used.

There is some flexibility in placing light standards. Whenever possible, place all light standards at the piers instead of in the spans for both aesthetics and vibration problems. Place 4' from pier if there is an expansion joint at the pier. WisDOT has experienced mast arm failures due to vibration on poles placed further from the pier.

With poles set in the center of the spans on bridges the heavy luminaire tends to stand still as the bridge deflects due to traffic. The pole shaft is too stiff to deflect much so the pole arm takes all the movement.

With a constant wind velocity the poles will vibrate. If they are placed too far into the span, deflections from traffic will induce further erratic vibrations. While single arm brackets are aesthetically appealing they are more prone to fatigue failures than the double arm brackets. Some single arm brackets have been replaced this way.

The resonant frequency of most poles is quite low (5 to 10 cycles per second). Therefore low wind velocities can excite these poles if they are not damped. In most cases the arm and luminaire do some of this. One case where this didn't work was corrected by putting vermiculite in the pole.

Some pole vibrations cause the bulbs to unscrew and fall out. This is corrected by attaching a clamp over the end of the bulb.

55 foot long poles with 20 foot mast arms can have a noticeable bend in the pole due to the dead load of the luminaire and mast arm up to approximately 12 inches.

The pole manufacturers suggest that the poles be manufactured with a curve so that the dead load of the arm and luminaire cause the centerline of the pole to approximate a straight line. They did not want to increase the pole cost by using more material. A fair tolerance should be allowed on the prescribed shape of the pole.

For high mast lighting questions, please contact the BOS ancillary structures maintenance engineer.

Table of Contents

41.7 References

1. *Specification for the National Bridge Inventory Bridge Elements Bridges* by Federal Highway Association, 2014

2. *Manual for Bridge Element Inspection, 2nd Edition* by American Association of State Highway Transportation Officials, 2019

3. *Structure Inspection Manual* by Wisconsin Department of Transportation, 2003.

4. *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* by Federal Highway Association, 1995

5. *Facilities Development Manual (FDM)* by Wisconsin Department of Transportation, 2018

6. *Program Management Manual (PMM)* by Wisconsin Department of Transportation, Division of Transportation Investment Management, 2018

This page intentionally left blank.

Table of Contents

42.1 Overview

This chapter provides goals, objectives, measures, and strategies for the preservation of bridges. This chapter contains criteria that is used to identify condition based and cyclical preservation, maintenance, and improvement work actions for bridges. Bridge preservation is defined as actions or strategies that prevent, delay, or reduce deterioration of bridges or bridge elements; restore the function of existing bridges; keep bridges in good or fair condition; and extend their service life. Preservation actions may be cyclic or condition-driven. (1)

A successful bridge program will seek a balanced approach to preservation, rehabilitation, and replacement. One measure of success is to maximize the life of structures while minimizing the life cycle cost. Preservation of structures is one of the strategies in maximizing the effectiveness of the overall bridge program by retarding the rate of overall deterioration of the bridges.

Bridges are key components of our highway infrastructure. Wisconsin has over 14,000 bridges, of which about 37% are owned by WisDOT. The average age of these bridges in 2019 is 38 years. The aging infrastructure is expected to deteriorate faster in the coming decades with increased operational demand unless concerted efforts are taken to preserve and extend their life. In addition, the state bridge infrastructure is also likely to see an increased funding competition among various highway assets. As a result, WisDOT must emphasize a concerted effort to preserve and extend the life of bridge infrastructure while minimizing long-term maintenance costs.

This chapter provides WisDOT personnel and partners with a framework for developing preservation programs and projects using a systematic and consistent process that reflects the environment and conditions of bridges and reflects the priorities and strategies of the Department.

A well-defined bridge preservation program will also help WisDOT use federal funding (2) for Preventative Maintenance (PM) activities by using a systematic process of identifying bridge preservation needs and its qualifying parameters as identified in FHWA's Bridge Preservation Guide (1). This chapter will promote timely preservation actions to extend and optimize the life of bridges in the state.

42.5.1 Eligibility Criteria

This chapter includes two distinct matrices outlining eligibility criteria for preservation activities shown in Table 42.5-2 and Table 42.5-3. The first matrix relates to concrete deck/slab activities and the second matrix covers other bridge component activities. Bridge inspection information and data that is managed in HSIS and the WISAMS (Chapter 41.2.1) will be used to develop reports that quantify needs at the program and project level. This method will also serve to develop reports to monitor progress related to performance goals.

The deck/slab matrix shown in Table 42.5-2 is based on the NBI Item 58 - Condition Rating for decks and total deck/slab distress area. The distress area on a deck is quantified using inspection defects including delaminations, spalls, cracking, and scaling. Other deck inspection methods such as chain drag sounding, ground penetrating radar (GPR) surveys, infrared (IR) surveys, and chloride potentials may also be used in quantifying deck defects.

The matrix shown in Table 42.5-3 is based on listed NBI condition ratings and specific inspection element condition states. As with decks, information and data from HSIS will be used with this matrix as well.

Table 42.5-3 also makes reference to “defects”. For a better understanding of this concept, the reader is referred to Appendix D of the *AASHTO Manual for Bridge Element Inspection*. This appendix describes the element materials defined for this guide and the defects that may be observed for each condition state. Included are individual materials, such as reinforced and prestressed concrete, steel, timber, masonry, and other materials.

These matrices guide the user to select a preservation activity and also show the potential enhancement to the NBI values and anticipated service life increase as a result of that activity. Note that even though some preservation activities list no change to the potential result to the condition rating of NBI items, there is an inherent benefit both in the short and long term of these preservation activities to extend the current condition and ultimately extend the life of the bridge.

Sound engineering judgment is needed to decide if the recommended action is best suited for extending the life of the bridge.

	NBI Item 58	Top Deck Element Distress Area (%)	Bottom Deck Element Distress Area (%)	Preservation Activity	Benefit to Deck from Action	Application Frequency (in years)
Concrete Deck/Slab	≥7	-	-	Deck Sweeping/Washing	Extend Service Life	1 to 2
		5% < 3220 < 25%	-	Crack Sealing	Extend Service Life	3 to 5
		3220 CS3 + CS4 > 0%	-	Deck Sealing	Service life extended	3 to 5
		-	1080 < 5%	Full Depth Deck Patching	Service life maintained	As needed
		3210 CS3 + CS4 < 5%	1080 < 5%	Wearing Surface Patching	Service life maintained	As needed
		>20% (3220 OR 8911 CS3 + CS4) OR	(1140 OR 1150) < 20% for timber deck	Polymer Modified Asphalt Overlay	Service life extended	10 to 15
		>15% 3210 (applied to bare deck)				
		>20% (3210 OR 8911 CS3 + CS4) OR	1080 < 5% for concrete deck			
		>50% 3220 (reapplication)				
		>20% (3220 OR 8911 CS3 + CS4) OR	(1140 OR 1150) < 20% for timber deck	HMA w/ membrane	Service life extended	5 to 15
		>15% 3210 (applied to bare deck)				
		>20% (3210 OR 8911 CS3 + CS4) OR	1080 < 5% for concrete deck			
		>50% 3220 (reapplication)				
		3210 < 5%	1080 < 1%	Polyester Polymer Concrete	Service life extended	20 to 30
		3210 < 2% (applied to bare deck)	1080 < 1%	Thin Polymer Overlay	Service life extended	7 to 15
		8513 CS3 + CS4 > 15% (reapplication)				
	6	-	-	Deck Sweeping/Washing	Extend Service Life	1 to 2
		5% < 3220 < 25%	-	Crack Sealing	Extend Service Life	3 to 5
		3220 CS3 + CS4 > 0%	-	Deck Sealing	Service life extended	3 to 5
		-	1080 < 5%	Full Depth Deck Patching	Service life maintained	As needed
		3210 CS3 + CS4 < 5%	1080 < 5%	Wearing Surface Patching	Service life maintained	As needed
		>20% (3220 OR 8911 CS3 + CS4) OR	(1140 OR 1150) < 20% for timber deck	Polymer Modified Asphalt Overlay	Improve NBI (58) ≥ 7	10 to 15
		>15% 3210 (applied to bare deck)				
		>20% (3210 OR 8911 CS3 + CS4) OR	1080 < 5% for concrete deck			
		>50% 3220 (reapplication)				
		>20% (3220 OR 8911 CS3 + CS4) OR	(1140 OR 1150) < 20% for timber deck	HMA w/ membrane	Improve NBI (58) ≥ 7	5 to 15
		>15% 3210 (applied to bare deck)				
		>20% (3210 OR 8911 CS3 + CS4) OR	1080 < 5% for concrete deck			
		>50% 3220 (reapplication)				
		8513 CS3 + CS4 > 15% (reapplication)	1080 < 1%	Thin Polymer Overlay	Service life extended	7 to 15
		>20% (3220 OR 8911 CS3 + CS4) OR	1080 < 5% OR 1130 CS3 + CS4 < 25%	Concrete Overlay	Improve NBI (58) ≥ 7	12 to 20
		>15% 3210 (applied to bare deck)				
		>20% (3210 OR 8911 CS3 + CS4) OR				
		>50% 3220 (reapplication)				
	5	5% < 3220 < 25%	-	Crack Sealing	Extend Service Life	3 to 5
		3220 CS3 + CS4 > 0%	-	Deck Sealing	Service life extended	3 to 5
		-	1080 < 5%	Full Depth Deck Patching	Service life maintained	As needed
		3210 CS3 + CS4 < 5%	1080 < 5%	Wearing Surface Patching	Service life maintained	As needed
		>20% (3220 OR 8911 CS3 + CS4) OR	1080 < 5% OR 1130 CS3 + CS4 < 25%	Concrete Overlay	Improve NBI (58) ≥ 7	12 to 20
		>15% 3210 (applied to bare deck)				
		>20% (3210 OR 8911 CS3 + CS4) OR				
		>50% 3220 (reapplication)				
	4	>20% (3220 OR 8911 CS3 + CS4) OR	1080 < 5% OR 1130 CS3 + CS4 < 25%	Concrete Overlay	Improve NBI (58) ≥ 7	12 to 20
		>15% 3210 (applied to bare deck)				
		>20% (3210 OR 8911 CS3 + CS4) OR				
		>50% 3220 (reapplication)				
	≤ 4	-	1080 > 15% OR 1130 CS3 + CS4 > 50%	Deck Replacement	Improve NBI (58) = 9	25 to 45

Table 42.5-2
Concrete Deck/Slab Eligibility Matrix

42.7 Definitions

Bridge Program: The WisDOT Bridge Program includes preservation, rehabilitation, improvement or major rehabilitation, replacement and new bridge construction actions.

Bridge Preservation: Bridge Preservation is defined as actions or strategies that prevent, delay, or reduce deterioration of bridges or bridge elements, restore the function of existing bridges, keep bridges in good or fair condition and extend their service life. Preservation actions may be cyclic or condition-driven.

Highway Structures Information System: Highway Structures Information System (HSIS) is the system developed by WisDOT for managing the inventory and inspection data of all highway structures. The inspection data is collected in accordance with the NBIS and *2019 AASHTO Manual for Bridge Element Inspection.*

Wisconsin Structures Asset Management System (WiSAMS): Automated application to determine optimal work candidates for improving the condition of structures. This application serves as a programming and planning tool for structures improvements, rehabilitations, maintenance, and preservation. This application coupled with the Highways Structures Information System (HSIS) serves as a comprehensive Structures (Bridge) Management system.

State of Good Repair (SGR): State of Good Repair (SGR) is a condition in which the existing physical assets, both individually and as a system (a) are functioning as designed within their useful service life, and (b) are sustained through regular maintenance and replacement programs. SGR represents just one element of a comprehensive capital investment program that also addresses system capacity and performance.[3]

Systematic Preventive Maintenance Program (SPM): Systematic Preventive Maintenance (SPM) is a planned strategy of cost-effective treatments to highway bridges that are intended to maintain or preserve the structural integrity and functionality of bridge elements and/or components, and retard future deterioration, thus maintaining or extending the useful life of bridges. An SPM program is based on a planned strategy that is equivalent to having a systematic process that defines the strategy, how it is planned, and how activities are determined to be cost effective. An SPM program may be applied to bridges at the network, highway system, or region-wide basis and have acceptable qualifying program parameters. The details on an SPM program and qualifying parameters are found in FHWA's *Bridge Preservation Guide.*

Preventive Maintenance(PM): Preventive maintenance is a cost-effective means to extend the service life of bridges. PM treatments retard future deterioration and avoid large expensive bridge rehabilitation or replacements. PM includes cyclic and condition based treatments.

Cyclical PM Activities: Cyclical PM activities are those activities performed on a pre-determined interval and aimed to preserve existing bridge element or component conditions. Bridge element or component conditions are not always directly improved as a result of these activities, however deterioration is expected to be delayed.

Condition Based PM Activities: Condition Based PM Activities are those activities that are performed on bridge elements in response to known defects as identified through the bridge inspection process.

Rehabilitation: Rehabilitation involves major work required to restore the structural integrity of a bridge as well as work necessary to correct major safety defects as defined in the Code of Federal Regulation (CFR) 23 clause 650.403.

Improvement or Major Rehab: Bridge improvement is a set of activities that fixes the deterioration found in a structure and improves the geometrics and load-carrying capacity beyond the original design standards, but may not provide improvement that meets new construction standards.

Replacement: Replacement of an existing bridge with a new facility constructed in the same general traffic corridor is considered total replacement. The replacement structure must meet the current geometric, construction, and structural standards as defined in the Code of Federal Regulation (CFR) 23 clause 650.403.

New Bridge Construction: The construction of a new bridge is defined as bridge construction that does not replace or relocate an existing bridge as described in FHWA's MAP-21 STP.

NBI Condition Rating: The FHWA coding guide describes the condition ratings used in evaluating four main components of a bridge as decks, superstructure, substructure, and culverts. The condition ratings are used to measure the deterioration level of bridges in a consistent and uniform manner to allow for comparison of the condition state of bridges on a national level. The condition ratings are also known as NBI ratings and are measured on a scale of 0 (worst) to 9 (excellent). For WisDOT bridges and culverts, an NBI rating of 4 is classified as poor, an NBI rating of 5 is classified as fair, and an NBI rating of 6 or higher is classified as 'good' (See Table 42.7-1).

Code	Description	Common Actions
9	**EXCELLENT CONDITION**	Preservation/Cyclic Maintenance
8	**VERY GOOD CONDITION**—No problems noted.	
7	**GOOD CONDITION**—Some minor problems.	
6	**SATISFACTORY CONDITION**—Structural elements show some minor deterioration.	Preservation/ Condition-Based Maintenance
5	**FAIR CONDITION**—All primary structural elements are sound but may have some minor section loss, cracking, spalling, or scour.	
4	**POOR CONDITION**—Advanced section loss, deterioration, spalling, or scour.	Rehabilitation or Replacement
3	**SERIOUS CONDITION**—Loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.	
2	**CRITICAL CONDITION**—Advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present, or scour may have removed substructure support. Unless closely monitored, the bridge may have to be closed until corrective action is taken.	
1	**IMMINENT FAILURE CONDITION**—Major deterioration or section loss present in critical structural components, or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic, but corrective action may put it back in light service.	
0	**FAILED CONDITION**—Out of service. Bridge is beyond corrective action.	

Table 42.7-1
NBI General Condition Ratings & Common Actions

Element Condition State: A condition state categorizes the nature and extent of damage or deterioration of a bridge element. The 2019 *AASHTO Manual for Bridge Element Inspection* describes a comprehensive set of bridge elements mainly categorized as National Bridge Elements (NBE), Bridge Management Elements (BME) and Agency Develop Elements (ADE) and their corresponding four condition states. The element condition states1 to 4 are described as good (CS1), fair (CS2), poor (CS3), and severe (CS4).

Condition State	Description	Common Actions[10]
1	Varies depending on element—Good	Preservation/Cyclic Maintenance
2	Varies depending on element—Fair	Cyclic Maintenance or Condition-Based Maintenance when cost effective.
3	Varies depending on element—Poor	Condition-Based Maintenance, or Rehabilitation—when quantity of poor exceeds a limit that condition-based maintenance is not cost effective, or Replacement—when rehabilitation is not cost effective.
4	Varies depending on element—Severe	Rehabilitation or Replacement

Table 42.7-2
Element Condition States & Common Actions

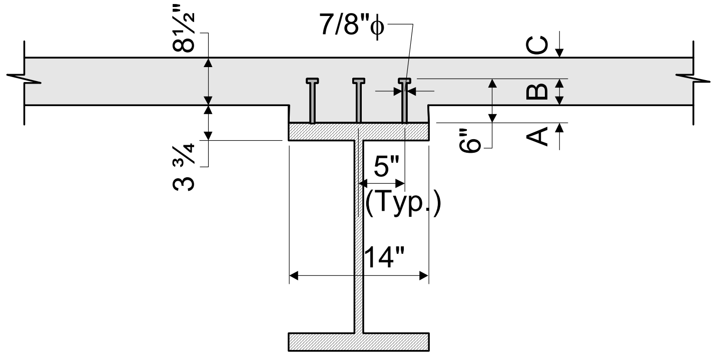


Figure E45-4.1-5
Composite Cross Section at Location of Maximum Positive Moment (0.4L)
(Note: 1/2" Intergral Wearing Surface has been removed for structural calcs.)

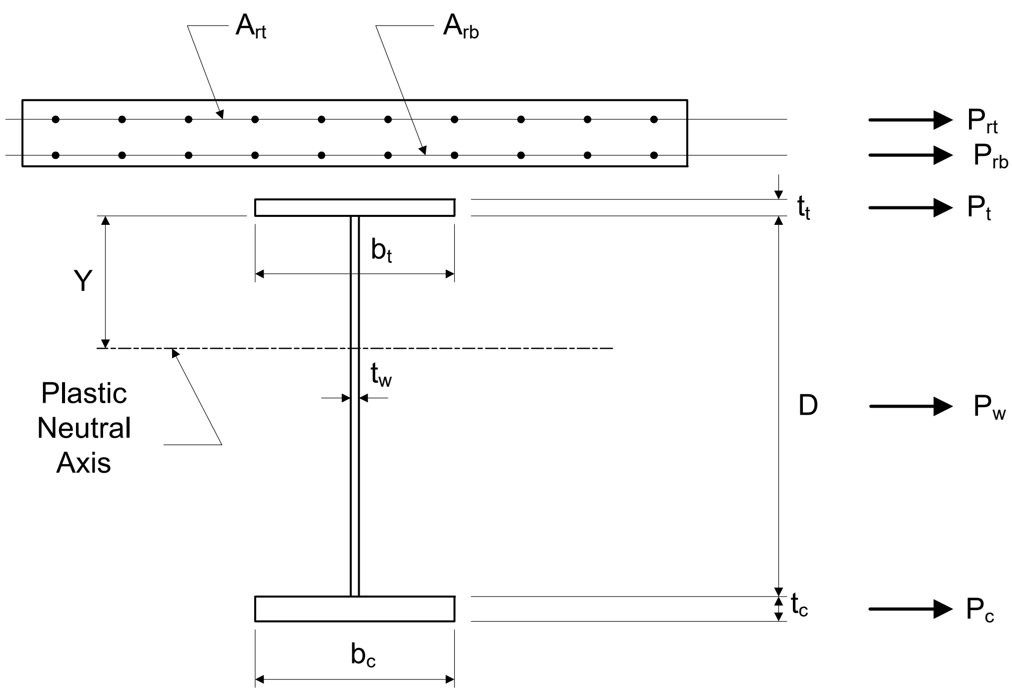


Figure E45-4.1-6
Composite Cross Section at Location of Maximum Negative Moment over Pier

$D := 54$ in

$t_w := 0.5$ in

E45-4.2 Compute Section Properties

Since the superstructure is composite, several sets of section properties must be computed **LRFD [6.10.1.1]**. The initial dead loads (or the noncomposite dead loads) are applied to the girder-only section. For permanent loads assumed to be applied to the long-term composite section, the long-term modular ratio of 3n is used to transform the concrete deck area **LRFD [6.10.1.1.1b]**. For transient loads assumed applied to the short-term composite section, the short-term modular ratio of n is used to transform the concrete deck area.

The modular ratio, n, is computed as follows:

$$n = \frac{E_s}{E_c}$$

Where:

E_s = Modulus of elasticity of steel (ksi)

E_c = Modulus of elasticity of concrete (ksi)

$E_s = 29000$ ksi **LRFD [6.4.1]**

$E_c = 33000 \cdot K_1 \cdot \left(w_c^{1.5}\right) \cdot \sqrt{f'_c}$ **LRFD [C5.4.2.4]**

Where:

K_1 = Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test, and as approved by the authority of jurisdiction

w_c = Unit weight of concrete (kcf)

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

$w_c = 0.15$ kcf **LRFD [Table 3.5.1-1 & C3.5.1]**

$f'_c = 4.00$ ksi

$K_1 := 1.0$ **LRFD [5.4.2.4]**

$E_c := 33000 \cdot K_1 \cdot \left(w_c^{1.5}\right) \cdot \sqrt{f'_c}$ $\boxed{E_c = 3834}$ ksi

$n := \frac{E_s}{E_c}$ $\boxed{n = 7.6}$ **LRFD [6.10.1.1.1b]**

Therefore, use: $n := 8$

The effective flange width is computed as follows .

For interior beams, the effective flange width is calculated as per **LRFD [4.6.2.6]**:

1. 12.0 times the average thickness of the slab, plus the greater of web thickness or one-half the width of the top flange of the girder:

$$b_{eff2} := \frac{12 \cdot t_s + \frac{14}{2}}{12}$$

This is no longer a valid criteria, however it has been left in place to avoid changing the entire example at this time.

$\boxed{b_{eff2} = 9.08}$ ft

2. The average spacing of adjacent beams:

$$b_{eff3} := S$$

$\boxed{b_{eff3} = 9.75}$ ft

Therefore, the effective flange width is:

$$b_{effflange} := \min(b_{eff2}, b_{eff3})$$

$\boxed{b_{effflange} = 9.08}$ ft

or

$\boxed{b_{effflange} \cdot 12 = 109.00}$ in

For this design example, the slab haunch is 3.75 inches throughout the length of the bridge. That is, the bottom of the slab is located 3.75 inches above the top of the web The area of the haunch is conservatively not considered in the section properties for this example.

Based on the plate sizes shown in Figure E45-4.1-4, the noncomposite and composite section properties for the positive moment region are computed as shown in the following table. The distance to the centroid is measured from the bottom of the girder.

Positive Moment Region Section Properties						
Section	Area, A (Inches^2)	Centroid, d (Inches)	A*d (Inches^3)	I_o (Inches^4)	A*y^2 (Inches^4)	I_{total} (Inches^4)
Girder only:						
Top flange	10.50	55.25	580.1	0.5	8441.1	8441.6
Web	27.00	27.88	752.6	6561.0	25.8	6586.8
Bottom flange	12.25	0.44	5.4	0.8	8576.1	8576.9
Total	49.75	26.90	1338.1	6562.3	17043.0	23605.3
Composite (3n):						
Girder	49.75	26.90	1338.1	23605.3	12293.9	35899.2
Slab	38.60	62.88	2427.2	232.4	15843.4	16075.8
Total	88.35	42.62	3765.3	23837.7	28137.3	51975.0
Composite (n):						
Girder	49.75	26.90	1338.1	23605.3	31511.0	55116.2
Slab	115.81	62.88	7281.7	697.3	13536.3	14233.6
Total	165.56	52.06	8619.8	24302.5	45047.3	69349.8
Section	y_{botgdr} (Inches)	y_{topgdr} (Inches)	$y_{topslab}$ (Inches)	S_{botgdr} (Inches^3)	S_{topgdr} (Inches^3)	$S_{topslab}$ (Inches^3)
Girder only	26.90	28.73	---	877.6	821.7	---
Composite (3n)	42.62	13.01	24.51	1219.6	3995.5	2120.7
Composite (n)	52.06	3.56	15.06	1332.0	19474.0	4604.5

Table E45-4.2-1
Positive Moment Region Section Properties

Similarly, the noncomposite and composite section properties for the negative moment region are computed as shown in the following table. The distance to the centroid is measured from the bottom of the girder **LRFD [6.6.1.2.1, 6.10.5.1, 6.10.4.2.1]**.

For the strength limit state, since the deck concrete is in tension in the negative moment region, the deck reinforcing steel contributes to the composite section properties and the deck concrete does not. However, per 45.6.3, only the top longitudinal mat of steel is used for rating purposes. Per the design example, the amount of longitudinal steel within the effective slab area is 6.39 in^2. This number will be used for the calculations below.