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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.

4.3 Primary Features

Superstructure Type and Shape

At highway speeds, highway structures are viewed from 300-500 feet away. The general shape of the bridge, with an emphasis on thinness, produces the most appealing structure. Given that there are realistic physical limitations on thinness (without resorting to anchored end spans or other costly measures), the designer has other options available to achieve the appearance of thinness such as:

- Larger overhangs to create better shadow lines.
- Horizontal recess on the backside of the parapet, which could be stained or left as plain concrete. Any parapet that is non-standard (either side) is considered Community Sensitive Design (CSD).
- Eliminate or minimize pedestals along the parapet. Such pedestals tend to break up the horizontal flow and make the superstructure appear top heavy. Pedestals, if desired, are better left on the wings to delineate the beginning or end of the bridge or to frame the bridge when viewed from below. If used on the superstructure, keep the pedestal size smaller and space apart far enough to avoid a top heavy appearance. See Chapter 30 – Railings for further guidance.
- Minimize vertical or patterned elements on the backside of the parapet as such elements tend to break up the horizontal flow. Rock formliner has become an overused aesthetic enhancement for the backside of parapets, as its use oftentimes does not fit the surroundings. Any parapet that is non-standard (either side) is considered CSD. See Chapter 30 – Railings for further guidance.
- Structure type should be based on economics, not aesthetics. Additional costs associated with a preferred structure type are considered CSD. Add-ons, such as false arches, etc. are considered CSD.

Abutment Type and Shape

Wing walls are the most visible portion of the abutment. Unless pedestrians are beneath a bridge, formliners or other aesthetic enhancements are not very visible and should be left off of the abutment front face, as these *treatments provide no additional aesthetic value.*

Pier Type and Shape

Pier shapes should be kept relatively simple and uncluttered. For highway grade separations, the end elevation of the pier is the view most often seen by the traveling public. For slower speed roads or where pedestrians travel beneath a bridge, the front pier elevation is also seen. For taller piers, such as those used for multi-level interchanges or water crossings, the entire 3D-view of the pier is readily seen and the pier shape is very important. For such piers, a clean, smooth flowing slender shape that clearly demonstrates the flow of forces from the superstructure to the ground is essential. External and internal (reentrant) corners on the pier/column shaft should be kept to a reasonable number. (Approximately 8 external, 4 internal maximum).

Grade and/or Skew

While grade and skew cannot be controlled by the bridge design engineer, these geometric features do affect bridge appearance. For example, a steep grade or pronounced vertical curve makes the use of a block type rustication an awkward choice. Horizontal blocks are typically associated with buildings and block buildings tend to have level roof lines. Cut stone form liners used on steep grades or pronounced vertical curves require excessive cutting of forms, which drives up price. Consideration of abutment height warrants more consideration when bridges are on steep grades, with a more exposed abutment face on the high end of the bridge producing a more balanced look.

Large skews tend to make piers longer as well as making the front elevation of the pier more visible to properties adjacent to the bridge. With larger skews, having more than one multi-columned pier can create a ‘forest’ of pier columns if the columns are too numerous. Try to maximize column spacing or use multiple hammerhead piers to help alleviate this effect. Abutment wings tend to be longer on the acute corners of bridges. Whatever aesthetic treatment is used needs to be appropriate for both the longer and shorter wings.

The design engineer should keep in mind that a bridge is never entirely seen at a 90-degree angle as depicted in a side elevation view. As the person viewing the bridge moves closer to the bridge the pier directly in front of them will be seen nearly as an end elevation of the pier, while adjacent piers will start to be viewed more as a pier side elevation. The ‘forest’ of columns starts to take effect, again, especially for wider bridges.

4.5 Aesthetics Process

The structural design engineer needs to be involved early in the aesthetic decision making process. BOS should have early representation on projects with considerable aesthetic concerns. Throughout this process it is important to remember that aesthetics is a concept, not a commodity – it is about a look, not about what can be added to a structure.

WisDOT policy item: For current statewide policy on aesthetic and/or decorative features (CSD), please see the *Program Management Manual* (PMM). See 4.3 for discussion on primary features such as shape and 4.9 for simple aesthetic concepts. The information below is current WisDOT policy. **Note: Any deviation from the standard details found in the WisDOT Bridge Manual regarding aesthetic features requires prior approval from BOS.**

Aesthetic and/or Decorative Amenities (non-Participating, or CSD Amenities)

- *All* formliner is considered CSD. This includes geometric patterns, vertical ribs, rock patterns, custom patterns/designs, etc.
- Stain
- Ornamentation, including city symbols, city names, etc. (City symbols, city names, memorial names, etc. are not allowed on the structures).
- Fencing, railing, or parapets not described below.
- Structure shapes not defined in 4.3 and 4.9 or the standard details.

Note: Future maintenance costs can be substantial when factoring in not only surface preparation and stain/paint, but planning, mobilization and maintenance of traffic required that is entirely attributable to the maintenance project. For example, re-staining of concrete, when all project costs are accounted for, often exceeds $20/SF.

Participating (non-CSD) Amenities

- Street Names: Street names recessed in the bridge parapet, and stained for visibility, are considered a participating amenity. The street name is considered an assistance to drivers. Having the name in the parapet removes the sign from the side of the road, which is considered a maintenance problem and safety hazard.

- Protective Fence: Any standard fencing from the Wisconsin Bridge Manual is considered a participating amenity. Additional costs for decorative fencing requested by the municipality will be included as a non-participating amenity. Fencing can be either galvanized or a duplex system of galvanized with a colored polymer-coating and/or paint. The polymer coating and/or paint is a nominal cost that provides a longer service life for the fence.

- Bridge Rail: Any standard railing from the Wisconsin Bridge Manual is considered a participating amenity as long as the railing is required for pedestrian and/or bicyclist protection. There is no discernable difference in cost between any of the standard railings. Paint is a nominal cost that provides longer service life for the railing.

- Bridge Parapet: Any standard parapet from the Wisconsin Bridge Manual is considered a participating amenity. The Vertical Face Parapet 'TX' may be used as a participating amenity as long as the parapet is required for pedestrian and/or bicyclist protection. There is no discernable difference in cost between the Type 'TX' and a shorter, plain concrete parapet with railing that is often used for pedestrian and/or bicyclist protection.

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 CSD 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

4.8 Resources on Aesthetics

The *Bridge Aesthetic Sourcebook* from AASHTO is a very good source of practical ideas for short and medium span bridges. The Transportation Research Board (TRB) Subcommittee on Bridge Aesthetics authored this document and it can be found on the following website: The final printing of this guide (noted in the References) is available through the AASHTO publication website:

4.9 Non-CSD Aesthetic Concepts

Standards 4.02-4.05 provide details for acceptable non-CSD funded aesthetic concepts. The three types (Type I, Type II and Type III) show a plain wing, a wing with a rustication trim line and a wing with a recessed panel, respectively. For each given wing type, one or two acceptable parapet and/or pier details are shown.

- Type I: Simple features utilizing a plain wing, standard parapet and minimal pier rustications. Type I is ideal for most rural and some urban applications.

- Type II: The wings utilize the same rustication trim line as the columns. The columns can have single or paired rustication trim lines. Single rustication lines can be used for 32-inch parapets and double rustication lines can be used for 42-inch parapets. Type II can be used in urban applications and other limited areas.

- Type III: Recessed panel wings and recessed panel columns, along with standard parapets, are to be used in urban settings, only.

Within a given corridor, only one Type should be chosen so as not to create a disharmonious experience for those driving the corridor.

The following pages show renderings of the various non-CSD aesthetic concepts.

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5.1 Factors Governing Bridge Costs

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

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

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

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

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

5.3 Contract Unit Bid Prices

Refer to FDM 19-5-5 when preparing construction estimates and use the following estimating tools:

- Bid Express
- AASHTOWare Project Estimator
- Estimating Tools website

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 2019 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	23	120,327	17,518,289	67.10	145.59
Reinf. Conc. Slabs (Flat)	44	69,664	11,879,548	70.13	170.53
Reinf. Conc. Slabs (Haunched)	10	43,057	6,148,879	100.04	142.81
Prestressed Box Girder	1	1,253	268,037	101.17	213.92

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	19	170,986	27,970,532	75.00	163.58
Reinf. Conc. Slabs (Haunched)	3	18,772	3,060,054	63.04	163.01
Steel Beams	1	7,964	1,522,389	95.77	191.16
Steel Plate Girders	3	130,986	30,430,624	144.97	232.32

Table 5.4-2
Grade Separation Structures

Box Culvert Type	No. of Culverts	Cost per Lin. Ft.
Single Cell	8	2,496
Twin Cell	5	3,392
Three Cell	1	3,283

Table 5.4-3
Box Culverts

Bridge Type	Cost
(none)	--

Table 5.4-4
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	7	17,195	2,490,957	144.87
MSE Panel Walls	27	85,496	10,517,536	123.02
Modular Walls	0	--	--	-
Precast Panel and Wire Faced	0	--	--	--
Soldier Pile Walls	3	6,290	1,378,911	219.22
Steel Sheet Pile Walls	1	1,940	92,512	47.69

Table 5.4-5
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.	0	--	--	--
Butterfly (2-Signs)	Conc. Col.	0	--	--	--
	1-Steel Col.	0	--	--	--
Cantilever	Conc. Col	0	--	--	--
	1-Steel Col	2	56	42,520	1,518
Cantilever Full Span	Conc. Col.	0	--	--	--
	1-Steel Col.	0	--	--	--
	2-Steel Col.	10	735.5	126,495	1,719.86
Full Span	1-Steel Col.	3	187	45,069	723.04
	2-Steel Col.	0	--	--	--

Table 5.4-6
Sign Structures

5.4.2 2020 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	28	236,564	35,597,272	70.46	150.48
Reinf. Conc. Slabs (Flat)	35	57,402	10,783,692	72.40	187.86
Reinf. Conc. Slabs (Haunched)	7	53,236	6,866,154	65.48	128.98
Prestressed Box Girder	2	9,050	2,694,672	157.15	297.75
Steel Plate Girders	1	19,076	5,258,732	120.51	275.67

Table 5.4-7
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	22	211,991	34,051,252	71.64	160.63
Reinf. Conc. Slabs (Flat)	1	2,179	379,028	62.35	173.95
Reinf. Conc. Slabs (Haunched)	1	5,563	870,732	43.94	156.52

Table 5.4-8
Grade Separation Structures

Box Culvert Type	No. of Culverts	Cost per Lin. Ft.
Single Cell	17	1,708
Twin Cell	1	2,073
Three Cell	0	--

Table 5.4-9
Box Culverts

5.4.3 2021 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	29	220,753	35,044,116	71.47	158.75
Reinf. Conc. Slabs (Flat)	51	76,036	15,497,984	76.94	203.82
Reinf. Conc. Slabs (Haunched)	10	46,682	7,340,768	70.37	157.25
Prestressed Box Girder	0	--	--	--	--
Buried Slabs	2	5,419	1,256,806	72.16	231.93
Steel Plate Girders	0	--		--	--

Table 5.4-10
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	244,240	37,780,465	73.38	154.69
Reinf. Conc. Slabs (Flat)	0	--	--	--	--
Reinf. Conc. Slabs (Haunched)	0	--	--	--	--

Table 5.4-11
Grade Separation Structures

5.4.4 2022 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	29	134,583	25,559,025	88.73	189.91
Reinf. Conc. Slabs (Flat)	53	79,248	17,397,862	85.21	219.54
Reinf. Conc. Slabs (Haunched)	6	49,138	9,413,541	88.63	191.57
Prestressed Box Girder	0	--	--	--	--
Buried Slabs	0	--	--	--	--
Steel Plate Girders	0	--	--	--	--

Table 5.4-12
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	8	81,829	13,443,218	78.36	164.28
Reinf. Conc. Slabs (Flat)	0	--	--	--	--
Reinf. Conc. Slabs (Haunched)	0	--	--	--	--

Table 5.4-13
Grade Separation Structures

5.4.5 2023 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	12	70,546	13,054,625	93.75	185.05
Reinf. Conc. Slabs (Flat)	36	67,796	15,075,049	86.82	222.36
Reinf. Conc. Slabs (Haunched)	4	13,032	3,208,985	79.85	246.24
Prestressed Box Girder	1	1,374	482,870	210.74	351.43
Buried Slabs	1	1,446	199,089	50.84	137.68
Steel Plate Girders	0	--	--	--	--

Table 5.4-14
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	6	78,611	12,600,448	72.67	160.29
Reinf. Conc. Slabs (Flat)	0	--	--	--	--
Reinf. Conc. Slabs (Haunched)	4	27,603	7,188,282	73.19	260.42

Table 5.4-15
Grade Separation Structures

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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.

7. Rehabilitation structure plans should include a list of work to be performed.

   SCOPE OF WORK (Example)

   - REMOVAL OF EXISTING CONCRETE OVERLAY AND PLACEMENT OF A NEW OVERLAY
   - RETROFIT EXISTING PARAPETS
   - SCOUR REPAIR
   - CONCRETE SURFACE REPAIR AT LOCATIONS DIRECTED BY ENGINEER

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

This is a list of agencies that may need to be coordinated with. There may be other stakeholders that require coordination. Consult FDM Chapter 5 for more details on coordination requirements.

- Federal Highway Administration (FHWA)

  For unique structures, a copy of the finalized preliminary structure plans is forwarded by the BOS Design Supervisor to FHWA Division Bridge Engineer for review.

- 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 deep from top to bottom and 17 inches long. A border line is provided on the sheet 5/8 inch from the left and right edges, and ¼ inch from top and bottom 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, or ghost lines.

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 righthand side and should be placed above the dimension lines. Notes and dimension text are approximately 0.06 inches high; view titles are approximately 0.10 inches high (based on a 11”x17” sheet). 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.

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.

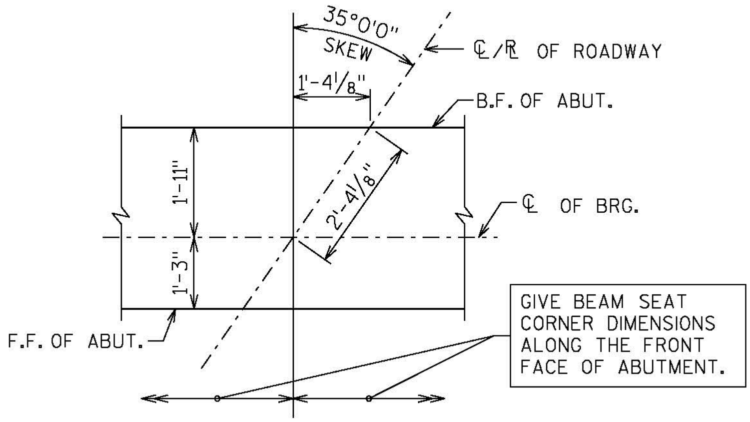

Figure 6.3-1
Example of Skewed Abutment Dimensions

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.

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 Site 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 responses to all preliminary plan comments. Responses to comments shall either agree or disagree with an explanation why the comment will not be incorporated in the final design and plans.
	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 Site 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. Final Site Investigation Report
4. Special Provisions covering unique items not in the Standard Specifications or Standardized Special Provisions (STSP).
5. QA/QC Verification Sheet
6. Inventory Data Sheet
7. Bridge Load Rating Summary Form
8. Responses to all BOS Preliminary Plan comments. Include responses (agree/disagree and why) on the marked up Preliminary Plan provided by BOS. Attach any additional clarifying notes or correspondence regarding Preliminary Plan comments to the end of the Preliminary Plan. E-submit as OTHER document.

9. 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.

6.5.7 Locally-Funded Projects

Local highway bridges designed under authority of the local municipality or county highway department without utilizing state or federal oversight must provide to BOS at minimum the following documents: (1) Bridge Inventory Data Sheet, (2) Scour assessment/evaluation documentation, (3) Bridge load rating summary sheet, and (4) Construction documentation such as an as-built plan and shop drawings. The scour assessment/evaluation documentation should be prepared in accordance with 8.3.2.7. All bridges shall be evaluated to determine the vulnerability to scour. See Chapter 8 – Hydraulics for additional guidance on hydrologic and hydraulic analysis.

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. Site Investigation 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 Summary sheet

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/

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11.1 General

11.1.1 Overall Design Process

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

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

2. Site Investigation Report – Based on the Structure Survey Report, a Geotechnical Investigation (see Chapter 10 – Geotechnical Investigation) is required, including test borings to determine foundation requirements. A hydraulic analysis is also performed at this time, if required, to assess scour potential and maximum scour depth. The Site Investigation Report and Subsurface Exploration Drawing are used to identify known constraints that would affect the foundations in regard to type, location or size and includes foundation recommendations to support detailed structural design. Certain structure sites/types may require the preliminary structure plans (Step 3) prior to initiating the geotechnical site investigation. One example of this is a multi-span structure over water. See 6.2 for more information.

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

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

11.1.2 Foundation Type Selection

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

- Magnitude and direction of loads.

- Depth to suitable bearing material.

- Potential for liquefaction, undermining or scour.

- Frost potential.

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

- Ease, time and cost of construction.

- Environmental impact of design and construction.

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

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

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

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

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

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

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

11.1.3 Cofferdams

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

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

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

Refer to 13.11.5 for additional information on cofferdams and seals.

11.1.4 Vibration Concerns

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

11.3 Deep Foundations

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

The primary functions of a deep foundation are:

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

11.3.1 Driven Piles

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

11.3.1.1 Conditions Involving Short Pile Lengths

WisDOT policy generally requires piles to penetrate a minimum of 10 feet through the original ground. Concern exists that short pile penetration in foundation materials of variable consistency may not adequately restrain lateral movements of substructure units. Pile penetrations of less than 10 feet are allowed if prebored at least 3 feet into solid rock. If conditions indicate that minimum pile penetration cannot be achieved, the preboring bid item should be included. Piles which are not prebored into rock must not only meet the 10-foot minimum pile penetration criteria but must also have at least 5 feet of penetration through material with a blow count of at least 7 blows per foot prior to meeting the required driving resistance. Refer to 11.3.1.6 for additional information on preboring.

Refer to 11.3.1.17.6 for additional information on scour considerations.

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

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

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

11.3.1.2 Pile Spacing

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

WisDOT policy item:

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

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

11.3.1.3 Battered Piles

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

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

11.3.1.4 Corrosion Loss

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

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

11.3.1.5 Pile Points

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

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

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

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

11.3.1.6 Preboring

If embedment into rock is required or minimum pile penetration is doubtful, preboring should be considered. It should be noted that preboring should only be used when appropriate, since many bridge contractors do not own the required construction equipment necessary for this work. Preboring is required for displacement piles when driven into new embankment with fill depths over 10 feet. For problem soils, contact the Bureau of Technical Services, Geotechnical Engineering Unit to discuss preboring considerations.

The following cases may warrant preboring:

- Displacement piles encountering a strong upper stratum with weak underlying soils. If soils (or consistent soil layers) that exhibit SPT refusal (e.g., 50 blows over 6 inches or less) are encountered prior to the scheduled pile tip elevation, pre-boring may be warranted to reduce the risk of unacceptably short pile lengths. Drivability analyses should consider harder than expected intermediate soil layers and be used to determine if preboring is warranted.

- Conditions involving short pile lengths, as discussed in 11.3.1.1. If embedment into rock is required or minimum pile penetration is doubtful, preboring should be considered. For short pile length conditions, piling should be prebored at least 3 feet into solid rock and “firmly seated” on rock after placement in prebored holes. The annular space between the cored rock holes and piling should then be filled with concrete.

Other preboring considerations:

- For displacement piles, preboring should be terminated at least 5 feet above the scheduled pile tip elevation.

- When the pile is planned to be point resistance on rock, preboring may be advanced to plan pile tip elevation. Piles placed in prebored holes founded on rock are typically firmly seated to promote firm contact between pile and rock and do not require driving or restrike to reduce the risk of pile damage.

- The annular space between the prebored hole and piling is required to be backfilled. After the pile is installed, concrete should be used to the top of the rock to properly socket point resistance piles. Clean sand should then be used to backfill the remaining annular space. Backfill material should be deposited with either a tremie pipe or concrete pump to reduce potential arching (bridging) and assure that the complete annular space is filled. Backfill materials for prebored holes should be clearly indicated in the plan documents.

- Some sites may require casing during the preboring operation. If casing is required, it should be clearly indicated in the plan documents.

See 11.3.1.17.6 for scour considerations.

lengths are a function of the assumed soil conditions and the required driving resistance. The as-built pile lengths are a function of the actual soil conditions encountered and the contractor's hammer selection.

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

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

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

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

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

<table>
<tr><th>Pier</th></tr>
<tr><td>Pier Example: 12 x 53 H-piles to an estimated length of 100 feet at a unit cost of $40/foot.

(Note: It is realized that for pier design the number of piles is not exclusively related to the vertical load, but this example is simplified for illustrative purposes).</td></tr>
<tr><td>Modified Gates:

RDR = 220 tons, FACR = 110 tons, Total Load on Pier = 3,500 tons, Number of Piles = 3,500 tons / 110 tons = 32 piles

<u>Pile Cost = 32 piles x 100 feet x $40/ft = $128,000</u>
Total Cost = $128,000</td></tr>
<tr><td>PDA/CAPWAP:</td></tr>
</table>

RDR = 220 tons, FACR = 143 tons, Total Load on Pier = 3,500 tons, Number of Piles = 3,500 tons / 143 tons = 25 piles Pile Cost = 25 piles x 100 feet x \$40/ft = \$100,000 PDA Testing Cost = 2 piles/sub. x \$700/pile = \$1,400 PDA Restrike Cost = 2 piles/sub. x \$600/pile = \$1,200 CAPWAP Evaluation = 1 eval./sub. x \$400/eval. = \$400 Total Cost = \$103,000
PDA/CAPWAP Savings = \$25,000/pier
Abutment
Abutment Example: 12 x 53 H-piles to an estimated length of 100 feet at a unit cost of \$40/foot.
Modified Gates: RDR = 220 tons, FACR = 110 tons, Total Load on Abut = 980 tons, Number of Piles = 980 tons / 110 tons = 9 piles Total Cost = 9 piles x 100 feet x \$40/ft = \$36,000
PDA/CAPWAP: RDR = 220 tons, FACR = 143 tons, Load on Abut = 980 tons, Number of Piles = 980 tons / 143 tons = 7 piles, however because of maximum spacing requirements the design will need 8 piles. Pile Cost = 8 piles x 100 feet x \$40/ft = \$32,000 PDA Testing Cost = 2 piles/sub. x \$700/pile = \$1,400 PDA Restrike Cost = 2 piles/sub. x \$600/pile = \$1,200 CAPWAP Evaluation = 1 eval./sub. x \$400/eval. = \$400 Total Cost = \$35,000
PDA/CAPWAP Cost = \$1000/abutment **Note:** For a three span bridge, with 12 x 53 H-piles to an estimated length of 100 feet at a unit cost of \$40/foot, PDA/CAPWAP would provide an estimated structure savings of \$52,000. For a two span bridge, with 12 x 53 H-piles to an estimated length of 40 feet at a unit cost of \$40/foot, PDA/CAPWAP would provide an estimated structure savings of \$5,400. Bid prices based on 2014-2015 cost data.

Table 11.3-6

Economical Evaluation for Deep Foundations with Two Construction Monitoring Methods

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12.12 MSE Walls at Abutments

MSE walls placed in front of pile supported abutments, as shown in Figure 12.12-1, may be an alternative to traditional sill abutments (Type A1) constructed at the top of the slope. This configuration shortens span lengths, which may be particularly beneficial in urban areas with limited right of way or other site constraints. However, use of MSE walls at abutments should be evaluated on a project-by-project basis. Historically, MSE walls have not been used for the singular purpose of reducing span length. See Chapter 14 - Standard Details for additional information.

Abutments supported entirely by MSE walls, also referred to as a “true MSE bridge abutment" and similar to GRS abutments, are prohibited.

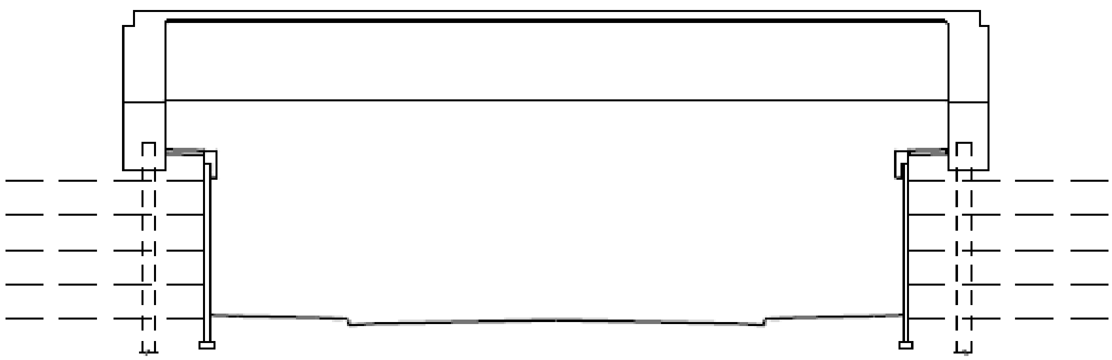

Figure 12.12-1
Sill Abutment with MSE Walls

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14.1 Introduction

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

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

WisDOT policy item:

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

14.1.1 Wall Development Process

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

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

Based on the SSR, a Geotechnical site investigation (see Chapter 10 – Geotechnical Investigation) may be required to determine foundation and retained soil properties. A hydraulic analysis is also conducted, if required, to asses scour potential. The Geotechnical investigation generally includes a subsurface and laboratory investigation. For the departmental-designed walls, the Bureau of Technical Services, Geotechnical Engineering Unit can recommend the scope of soil exploration needed and provide/recommend bearing resistance, overall stability, and settlement of walls based on the geotechnical exploration results. These Geotechnical recommendations are presented in a Site Investigation Report.

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

Proprietary walls, as defined in 14.2, are pre-approved by the WisDOT's Bureau of Structures. Preapproval process for the proprietary walls is explained in 14.16. The structural design, internal and final external stability of proprietary wall systems are the responsibility of the supplier/contractor. The design and shop drawing computations of the proprietary wall systems are also reviewed by the Bureau of Structures in accordance with the plans and special provisions. The preliminary external stability, overall stability and settlement computations of these walls are performed by the Geotechnical Engineering Unit or the WisDOT's Consultant in the project design phase. Design and shop drawings must be accepted by the Bureau of Structures prior to start of the construction. Design of all temporary walls is the responsibility of the contractor.

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

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

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

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

14.1.1.1 Wall Numbering System

Refer to 2.5 for assigning structure numbers.

14.6 Mechanically Stabilized Earth Retaining Walls

14.6.1 General Considerations

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

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

The MSE walls shall be designed for external stability of the wall system and internal stability of the reinforced soil mass. The global stability shall also be considered as part of design evaluation. MSE walls are proprietary wall systems and the design responsibilities with respect to global, external, and internal stability as well as settlement are shared between the designer (WisDOT or Consultant) and contractor. The designer is responsible for the overall stability, preliminary external stability and settlement whereas the contractor is responsible for the internal stability, compound stability and structural design of the wall. For settlement, the designer shall select the appropriate wall facing type (e.g. small 5'x5' precast panels) and locate slip joints locations, as required. The contractor should accommodate wall settlement shown on contract documents and based on the wall supplier recommendations. The responsibilities of the designer and contractor are outlined in 14.6.3.2. The design and drawings of MSE walls provided by the contractor must also be in compliance with the WisDOT special provisions as stated in 14.15.2 and 14.16.

The design engineer should detail the MSE wall and any supporting structures (e.g. a bridge abutment) to ensure settlements are properly accommodated. This may include limiting the MSE wall to small precast concrete panels (<30 sf ft), detailing coping extensions on adjacent structures, or locating ship joints accordingly.

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

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

14.6.1.1 Usage Restrictions for MSE Walls

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

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

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

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

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

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

14.6.2 Structural Components

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

- Selected Earthfill in the Reinforced Earth Zone
- Reinforcement
- Wall Facing Element
- Leveling Pad
- Wall Drainage

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

The non-metallic or extensible reinforcement includes the following:

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

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

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

14.6.2.3 Facing Elements

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

Major facing types are:

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

Segmental Precast Concrete Panels

Segmental precast concrete panels include small panels (<30 sq ft) to larger (≥30 sq ft and < 75 sq ft) with a minimum thickness of 5-½ inches and square or rectangular in geometry. Less common geometries such as cruciform, diamond, and hexagonal are currently not being used. The geometric pattern of the joints and the smooth uniform surface finish of the factory provided precast panels give an aesthetically pleasing appearance. Segmental precast concrete panels are proprietary wall components.

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

WisDOT requires that MSE walls utilize precast concrete panels when supporting traffic live loads which are in close proximity to the wall. Panels are also allowed as components of an abutment structure. Either steel strips or welded wire fabric is allowed for soil reinforcement when precast concrete panels are used as facing of the MSE wall system.

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

Concrete Modular Blocks Facings

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

Figure 14.6-3 shows a typical modular block MSE wall system along with other wall components. Most modular block MSE walls are reinforced with geogrids.

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

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

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

Alignments that are not straight (i.e. kinked or curved) shall use 90 degree corners or curves. The minimum radius should be limited to 8 feet. For a concave wall the radius is measured to the front face of the bottom course. For convex walls the radius is measured to the front face

14.12 Temporary Shoring

This information is provided for guidance. Refer to the *Facilities Development Manual* for further details.

Temporary shoring is used to support a temporary excavation or protect existing transportation facilities, utilities, buildings, or other critical features when safe slopes cannot be made for structural excavations. Shoring may be required within the limits of structures or on the approach roadway due to grade changes or staged construction. Temporary shoring generally includes non-anchored temporary sheet piles, temporary soldier pile walls, temporary soil nails, cofferdam, or temporary mechanically stabilized earth (MSE) walls.

Temporary shoring is designed by the contractor. Shoring should not be required nor paid for when used primarily for the convenience of the contractor.

14.12.1 When Slopes Won't Work

Typically shoring will be required when safe slopes cannot be made due to geometric constraints of existing and proposed features within the available right-of-way. Occupation and Healthy Safety Administration (OSHA) requirements for temporary excavation slopes vary from a 1H:1V to a 2H:1V. The contractor is responsible for determining and constructing a safe slope based on actual site conditions.

In most cases, the designer can assume that an OSHA safe temporary slope can be cut on a 1.5H:1V slope; however other factors such as soil types, soil moisture, surface drainage, and duration of excavation should also be factored into the actual slope constructed. As an added safety factor, a 3-foot berm should be provided next to critical points or features prior to beginning a 1.5H:1V slope to the plan elevation of the proposed structure. Sufficient room should be provided adjacent to the structure for forming purposes (typically 2-3 feet).

14.12.2 Plan Requirements

Contract plans should schematically show in the plan and profile details all locations where the designer has determined that temporary shoring will be required. The plans should note the estimated length of the shoring as well as the minimum and maximum required height of exposed shoring. These dimensions will be used to calculate the horizontal projected surface area projected on a vertical plane of the exposed shoring face.

14.12.3 Shoring Design/Construction

The Contractor is responsible for design, construction, maintenance, and removal of the temporary shoring system in a safe and controlled manner. The adequacy of the design should be determined by a Wisconsin Professional Engineer knowledgeable of specific site conditions and requirements. The temporary shoring should be designed in accordance with the requirements described in 14.4.2 and 14.4.3. A signed and sealed copy of proposed designs must be submitted to the WisDOT Project Engineer for information.

14.13 Noise Barrier Walls

14.13.1 Wall Contract Process

WisDOT has classified all noise walls (both proprietary and non-proprietary) into three wall systems. All proprietary systems must be pre-approved prior to being considered for use on WisDOT projects. The three noise wall systems that are considered for WisDOT projects include the following:

1. Double-sided sound absorptive noise barriers
2. Single-sided sound absorptive noise barriers
3. Reflective noise barriers

If a wall is required, the designer must determine which wall system or systems are suitable for a given wall location. In some locations all wall systems may be suitable, whereas in other locations some wall systems may not be suitable. Information on aesthetic qualities and special finishes and colors of proprietary systems is available from the manufacturers. Information on approved concrete paints, stains and coatings is also available from the Structures Design Section. Designers are encouraged to contact the Bureau of Structures Design Section if they have any questions about the material presented in the *Bridge Manual*.

The step by step process required to select a suitable wall system or systems for a given wall location is as follows:

Step 1: Investigate alternatives

Investigate alternatives to walls such as berms, plantings, etc.

Step 2: Geotechnical analysis

If a wall is required, geotechnical personnel shall conduct a soil investigation at the wall location and determine soil design parameters for the foundation soil. Geotechnical personnel are also responsible for recommending remedial methods of improving soil bearing capacity if required.

Step 3: Evaluate basic wall restrictions

The designer shall examine the list of suitable wall systems using the Geotechnical Report and remove any system that does not meet usage restrictions for the site.

Step 4: Determine suitable wall systems

The designer shall further examine the list of suitable wall systems for conformance to other considerations. Refer to Chapter 2 – General and Chapter 6 – Plan Preparation for a discussion on aesthetic considerations.

Step 5: Determine contract letting

14.17 References

1. State of Wisconsin, Department of Transportation, *Facilities Development Manual*

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

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

4. AASHTO LRFD Bridge Design Specification 4th Edition, 2007, AASHTO, 444 North Capitol Street, N.W., Suite 249, Washington, D.C. 20001.

5. Bowles, Joseph E*. Foundation Analysis and Design 4th Edition.* McGraw Hill *1989*

6. Cudoto, Donald P. *Foundation Design Principles and Practices (* 2nd Edition*),* Prentice Halls

7. National Concrete Masonry Association, "Design Manual for Segmental Retaining Walls", 2302 Horse Pen Road, Herndon, Virginia 22071-3406.

8. Publication No. FHWA-NHI-14-007, “Geotechnical Engineering Circular No. 7 Soil Nail Walls – Reference Manual”

9. Publication No. FHWA-SA-96-069R, “Manual for Design and Construction of Soil Nail walls”

10. Publication No. FHWA-HI-98-032, "Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures”

11. Publication No. FHWA-NHI-07-071, "Earth Retaining Structures”

12. Publication No. FHWA-NHI-09-083, "Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures”

13. Publication No. FHWA-NHI-09-087, "Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced slopes"

14. Publication No. FHWA-NHI-10-024, "Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes-Volume I”

15. Publication No. FHWA-NHI-10-025, "Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes-Volume II”

16. Publication No. FHWA-NHI-24-002, "Design and Construction of Mechanically Stabilized Earth (MSE) Walls”

14.18 Design Examples

E14-1	Cast-In-Place Concrete Cantilever Wall on Spread Footing, LRFD
E14-2	Precast Panel Steel Reinforced MSE Wall, LRFD
E14-3	Modular Block Facing Geogrid Reinforced MSE Wall, LRFD
E14-4	Cast-In-Place Concrete Cantilever Wall on Piles, LRFD
E14-5	Sheet Pile Wall, LRFD

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18.1 Introduction

18.1.1 General

This chapter considers the following types of concrete structures:

- Flat Slab
- Haunched Slab

A longitudinal slab is one of the least complex types of bridge superstructures. It is composed of a single element superstructure in comparison to the two elements of the transverse slab on girders or the three elements of a longitudinal slab on floor beams supported by girders. Due to simplicity of design and construction, the concrete slab structure is relatively economical. Its limitation lies in the practical range of span lengths and maximum skews for its application. For longer span applications, the dead load becomes too high for continued economy. Application of the haunched slab has increased the practical range of span lengths for concrete slab structures.

18.1.2 Limitations

Concrete slab structure types are not recommended over streams where the normal water freeboard is less than 4 feet; formwork removal requires this clearance. When spans exceed 35 feet, freeboard shall be increased to 5 feet above normal water.

All concrete slab structures are limited to a maximum skew of 30 degrees. Slab structures with skews in excess of 30 degrees, require analysis of complex boundary conditions that exceed the capabilities of the present design approach used in the Bureau of Structures.

Continuous span slabs are to be designed using the following pier types:

- Piers with pier caps (on columns or shafts)
- Wall type piers

These types will allow for ease of future superstructure replacement. Piers that have columns without pier caps, have had the columns damaged during superstructure removal. This type of pier will not be allowed without the approval of the Structures Design Section.

WisDOT policy item:

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

The following is a list of items that do not need to be submitted as a part of the final e-submittal to BOS for review:

- Design Computations (unless there is a unique design feature)
- Bridge Load Rating Summary Form

18.5.4.2 Location of Tool

The SBDT is a web-based application that can be found at the following location:

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

18.5.4.3 How to Utilize the Tool

The step-by-step user guide can be found at the following location:

https://wisconsindot.gov/Pages/doing-bus/local-gov/lpm/lp-standarized-bridge-plan-pilot.aspx.

18.6 Design Example

E18-1 Continuous 3-Span Haunched Slab, LRFD

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28.1 Introduction

Many structures have joints that must be properly designed and installed to insure their integrity and serviceability. Bridges as well as highway pavements, airstrips, buildings, etc. need joints to take care of expansion and contraction caused by temperature changes. However, bridges expand and contract more than pavement slabs or buildings and have their own special types of expansion devices.

Current practice is to limit the number of bridge expansion joints. This practice results in more movement at each joint. There are so many potential problems associated with joints that fewer joints are recommended practice. Expansion joints are placed on the high end of a bridge if only one joint is placed on the bridge. This is done to prevent the bridge from creeping downhill and to minimize the amount of water passing over the joint.

Open joints generally lead to future maintenance. Water and debris fall through the joint. Water running through an open joint erodes the soil under the structure, stains the bent cap and columns, and leads to corrosion of adjacent girders, diaphragms, and bearings. During freeze-thaw conditions, large icicles may form under the structure or ice may form on the roadway presenting a traffic hazard. Debris acts with water in staining the substructure units and plugs the drainage systems.

In the past, open steel finger type joints were used on long span bridges where large movements encountered. Finger joints were placed in the span near the point of contraflexure and were placed on the structure where they are required structurally. Drains were located to prevent drainage across the joint if feasible. In some areas, they were provided with a drainage trough to collect the water passing through.

Sliding steel plate joints are semi-open joints since water and light debris can pass through. A sealant placed in the joint prevents some water from passing through. It also prevents the accumulation of debris which can keep the joint from moving as it was designed. To date, considerable maintenance has occurred with sealants and neoprene troughs have been added to collect the water at some sites.

Currently finger and sliding plate details are maintained for joint maintenance and retrofitting but are not used for new structures. Watertight expansion devices such as strip seals and modular types are recommended for new structures. Although these expansion joints are not completely watertight; they have been effective in reducing damage to adjacent girders, diaphragms, bearings and substructure units.

The neoprene compression seal is a closed joint which is watertight if it is properly installed and an adequate adhesive is employed. Compression seals are only used for fixed joints. Strip-seals are watertight joints which are used in place of sliding plate joints or finger joints in an attempt to keep water and debris on the bridge deck surface.

Refer to Figure 12.7-1 for placement of expansion devices. Criteria for placement of expansion devices is described in the following sections.

28.1.1 General

Use watertight expansion joints wherever possible according to the design criteria and of all structure lengths. On skews over 45°, strip seals must be oversized to compensate for racking of the joint. For thermal movements greater than 4 inches modular expansion devices are recommended.

28.1.2 Concrete Spans

An expansion device is required if the expansion length of the structure exceeds 300 feet. At this point the geometrics of the structure determine the number of expansion joints required with a maximum expansion length of 400 feet.

As an example, consider a prestressed girder structure 700 feet long on flexible piers and 0° skew. Considering the two piers near the center of the span as fixed, the structure can expand toward each abutment with maximum expansion lengths less than 400 feet. A 400 series model strip seal expansion joint at each abutment is adequate for this structure.

28.1.3 Steel Spans

Watertight joints are required on all painted and unpainted steel structures to control staining of the substructure units due to corrosion of the steel girders, diaphragms, and bearings.

See Figure 12.7-1 to determine the appropriate abutment type and, hence, whether expansion devices are required. The geometry of the structure determines the number of expansion devices required and the amount of movement at each device. Some factors to consider are temperature expansion with high skew angles may cause "racking" of the structure; higher abutments have more uncertainty to movement due to backfill pressure; and curved girders add torsional and shear forces.

Long span structures on tall flexible piers may have much longer expansion lengths than short span structures on short rigid piers. The longer spans have much less resistance to horizontal temperature movement caused by bearing friction and pier rigidity. These types of structures are designed for joint movements of 4 inches or greater using modular expansion devices.

28.1.4 Thermal Movement

The maximum thermal movement required at expansion joints is based on the following table:

Structure Type	Temperature Range	Thermal Coefficient
Steel:	-30 to 120°F	0.0000065/F
Concrete:	+5 to 85°F	0.0000060/F
*Prestressed Girder:	+5 to 85°F	0.0000060/F

Table 28.1-1
Thermal Movement

* For Prestressed girders add shrinkage due to creep of .0003 ft/ft. This value should be used in setting the joint opening as the joint opening will continue to widen over time.

The expansion length is measured along the centerline of the bridge and the length is normal to the joint opening for structures with a zero skew. The designer should provide adequately sized joints (i.e. round up in size if between two joint sizes or use additional joints or a different type of joint).

The annual mean temperature for Wisconsin is 45 °F. For the setting of strip seal expansion devices, see Standard for Strip Seal Expansion Joint Details for the joint opening when the expansion length is less than or equal to 230 feet. When the expansion length is greater than 230 feet show a temperature table with the joint openings from 5°F to 85°F in 10°F increments.

Note that the neutral point for temperature movement is not always located at the fixed pier. See Chapter 13 – Piers for an explanation of how to calculate the neutral point.

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30.1 Crash-Tested Bridge Railings and FHWA Policy

Notice: All contracts with a letting date after December 31, 2019 must use bridge rails and transitions meeting the 2016 Edition of MASH criteria for new permanent installation and full replacement.

WisDOT policy item:

For all Interstate structures, the 42SS parapet shall be used. For all STH and USH structures with a posted speed >= 45 mph, the 42SS parapet shall be used.

The timeline for implementation of the above policy is:

- All contracts with a letting date after December 31, 2019.
  (This is an absolute, regardless of when the design was started.)

- All preliminary designs starting after October 1, 2017
  (Even if the let is anticipated to be prior to December 31, 2019.)

Contact BOS should the 42" height adversely affect sight distance, a minimum 0.5% grade for drainage cannot be achieved, or for other non-typical situations.

Crash test procedures for full-scale testing of guardrails were first published in 1962 in the *Highway Research Correlation Services Circular 482*. This was a one-page document that specified vehicle mass, impact speed, and approach angle for crash testing and was aimed at creating uniformity to traffic barrier research between several national research agencies.

In 1974, the National Cooperative Highway Research Program (NCHRP) published their final report based on NCHRP Project 22-2, which was initiated to address outstanding questions that were not covered in *Circular 482*. The final report, NCHRP Report 153 – *"Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances,"* was widely accepted following publication; however, it was recognized that periodic updating would be required.

NCHRP Project 22-2(4) was initiated in 1979 to address major changes to reflect current technologies of that time and NCHRP Report 230, *"Recommended Procedures for the Safety Performance Evaluation of Highway Safety Appurtenances,"* was published in 1980. This document became the primary reference for full-scale crash testing of highway safety appurtenances in the U.S. through 1993.

In 1986, the Federal Highway Administration (FHWA) issued a policy memorandum that stated highway bridges on the National Highway System (NHS) and the Interstate Highway System (IHS) must use crash-tested railings in order to receive federal funding.

The American Association of State Highway and Transportation Officials (AASHTO) recognized that the evolution of roadside safety concepts, technology, and practices necessitated an update to NCHRP Report 230 approximately 7 years after its adoption. NCHRP Report 350, *"Recommended Procedures for the Safety Performance Evaluation of Highway Features,"* represented a major update to the previously adopted report. The updates

The application of bridge railings shall comply with the following guidance:

1. All bridge railings shall conform to **MASH 2016 requirements for lets after December 31, 2019**.

2. Traffic Railings placed on state-owned and maintained structures (Interstate Highways, United States Highways, State Trunk Highways, and roadways over such highways) with a design speed exceeding 45 mph shall be solid concrete parapets. Where the minimum 0.5% deck grade cannot be accommodated for proper drainage based on project specific constraints, the designer shall utilize open railings as described in this section. (**NOTE**: WisDOT does not currently have an open rail meeting the minimum MASH TL-3 requirements for NHS roadways or non-NHS roadways with design speeds exceeding 45 mph. An open rail meeting MASH TL-3 is being investigated.).

   Traffic Railings placed on locally-owned and maintained structures (County Trunk Highways, Local Roadways) with a design speed exceeding 45 mph are strongly encouraged to utilize solid concrete parapets.

3. Traffic Railings placed on structures with a design speed of 45 mph or less can be either solid concrete parapets or open railings with the exception as noted below in the single slope parapet application section. It should be noted that open railing bridges can incur maintenance issues with salt-water runoff over the edge of deck.

4. New bridge plans utilizing concrete parapets shall be designed with single-sloped (“SS”) parapets. See item No. 1 below for usage.

5. Per **LRFD [13.8.1]** and **LRFD [13.9.2]**, the minimum height of a Pedestrian (and/or bicycle) Railing shall be 42” measured from the top of the walkway or riding surface respectively. Per the *Wisconsin Bicycle Facility Design Handbook*, on bridges that are signed or marked as bikeways and bicyclists are operating right next to the railing, the preferred height of the railing is 54”. The higher railing/parapet height is especially important and should be used on long bridges, high bridges, and bridges having high bicyclist volumes. If an open railing is used, the clear opening between horizontal elements shall be 6 inches or less.

6. Aesthetics associated with bridge railings shall follow guidance provided in 30.4.

7. For bridge railings on un-posted roadways, assume a design speed limit of 55 mph for determining the appropriate bridge railing.

The designation for railing types are shown on the Standard Details. Bridge railings shall be employed as follows:

1. <u>The default parapet shall be the “42SS”</u>. If site distance issues arise due to the 42-inch height, please contact BOS for consideration of a shorter parapet (“32SS”and “36SS”). Single slope parapet “56SS” shall only be used if 56” CBSS adjoins the bridge. The “42SS” is TL-4 under MASH. The “32SS” is TL-3 under MASH. The “36SS” is TL-4 under MASH. *At this time, the “56SS” Test Loading is still unknown.*

A “SS” or solid parapet shall be used on all grade separation structures and railroad crossings to minimize snow removal falling on the traffic below.

2. The sloped face parapet "LF" and “HF” parapets shall be used as Traffic Railings for rehabilitation projects (joint repair, impact damage, etc.) only to match the existing parapet type. The sloped face parapets were crash-tested per NCHRP Report 230 and meet NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

3. The “51F” parapet shall only be used as a Traffic Railing on the median side of a structure when it provides a continuation of an approach 51 inch high median barrier.

4. Although the vertical face parapet “A” can be used for all design speeds, Bureau of Structures Development Section approval is required for design speeds exceeding 45mph. The vertical face parapet is recommended for use as a Combination Railing on raised sidewalks or as a Traffic Railing where the design speed is 45 mph or less. If the structure has a raised sidewalk on one side only, a sloped parapet should be used on the side opposite of the sidewalk. For design speeds exceeding 45 mph, at locations where the parapet is protected by a Traffic Railing between the roadway and a sidewalk at grade, the vertical face parapet can be used as a Pedestrian Railing. The vertical face parapet “A” is considered at TL-3 when on a bridge deck and TL-2 when on a raised sidewalk (The structural capacity is TL-3, however the vaulting effect of the sidewalk lowers the rating to TL-2).

5. Aesthetic railings may be used if crash tested according to 30.1 or follow the guidance provided in 30.4. See Chapter 4 – Aesthetics for CSD considerations.

   The Texas style aesthetic parapet, type “TX”, can be used as a Traffic/Pedestrian Railing on raised sidewalks on structures with a design speed of 45 mph or less. For design speeds exceeding 45 mph, at locations where the parapet is protected by a Traffic Railing between the roadway and a sidewalk at grade, the type “TX” parapet can be used. The type “TX” parapet is TL-2 under MASH.

6. The type “PF” tubular railing, as shown in the Standard Details of Chapter 40, shall not be used on new bridge plans with a PS&E after 2013. This railing was not allowed on the National Highway System (NHS). The type “PF” railing was used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less.

7. Combination Railings, type “C1” through “C6”, are shown in the Standard Details and are approved as aesthetic railings attached to concrete parapets. The aesthetic additions are placed at least 5” from the crash-tested rail face per the Standard Details and have previously been determined to not present a snagging potential. Combination railing, type “3T”, without the recessed details on the parapet faces may be used when aesthetic details are not desired or when CSD funding is not available (see Chapter 4 – Aesthetics). These railings can only be used when the design speed is 45 mph or less, or the railing is protected by a Traffic Railing between the roadway and a sidewalk at grade. The crash test criteria of the combination railings are based on the concrete parapets to which they are attached.

8. Chain Link Fence and Tubular Screening, as shown in the Standard Details, may be attached to the top of concrete parapets as part of a Combination Railing or as a Pedestrian Railing attached directly to the deck if protected by a Traffic Railing between the roadway and a sidewalk at grade. Due to snagging and breakaway potential of the vertical spindles, top-mounted Tubular Screening and Chain Link Fence should only be used when the design speed is 45 mph or less, or the screening is protected by a Traffic Railing between the roadway and a sidewalk at grade.

   Contact the Bureau of Structures Development Section when protective screening is warranted and used for design speeds exceeding 45 mph. In some cases, a Chain Link Fence mounted on the outside face (side-mounted) of the concrete parapet may be acceptable.

9. Type "H" aluminum or steel railing can be used on top of either vertical face or single slope parapets (“A” or “SS”) as part of a Combination Railing when required for pedestrians and/or bicyclists. For a design speed greater than 45 mph, the single slope parapet is recommended. Per the Standard Specifications, the contractor shall furnish either aluminum railing or steel railing. In general, the bridge plans shall include both options. For a specific project, one option may be required. This may occur when rehabilitating a railing to match an existing railing or when painting of the railing is required (requires steel option). If one option is required, the designer shall place the following note on the railing detail sheet: “Type H (insert railing type) railing shall not be used”. The combination railing is TL-3 under MASH.

10. Timber Railing as shown in the Standard Details is not allowed on the National Highway System (NHS). Timber Railing may be used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less. The Timber Railing has not been rated under MASH.

11. The type "W" railing, as shown in the Standard Details, is not allowed on the National Highway System (NHS). This railing may be used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less. The type “W” railing shall be used on concrete slab structures only. The use of this railing on girder type structures has been discontinued. Generally, type "W" railing is considered when the roadway approach requires standard beam guard and if the structure is 80 feet or less in length. Although the type “W” railing was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-3 (based on a May 1997 FHWA memorandum), FHWA has since restricted its use as indicated above.

12. Type “M” steel railing, as shown in the Standard Details, shall generally be used as a Traffic Railing on all functional classes of Wisconsin structures with a design speed of 45 mph or less. The type “M” railing may be used on roadways with a design speed exceeding 45 mph where the minimum 0.5% deck grade cannot be accommodated for proper drainage based on project specific constraints. The type “M” railing also can be used in place of the type “W” railing when placed on girder type structures as type “W” railings are not allowed for this application. However, the type “M” railing is not allowed for use on prestressed box girder bridges. This railing shall be considered where the Region requests an open railing. The type “M” railing is TL-2 under MASH.

13. Type “NY3/NY4” steel railings, as shown in the Standard Details, shall generally be used as a Traffic Railing on all functional classes of Wisconsin structures with a design speed of 45 mph or less. The type “NY3/NY4” railings may be used on roadways with a design speed exceeding 45 mph where the minimum 0.5% deck grade cannot be accommodated for proper drainage based on project specific constraints. The type “NY3/NY4” railings also can be used in place of the type “W” railing when placed on girder type structures as type “W” railings are not allowed for this application. The type “NY4” railing may be used on a raised sidewalk where the design speed is 45 mph or less. However, the type “NY” railings are not allowed for use on prestressed box girder bridges. These railings shall be considered where the Region requests an open railing. The type “NY” railings are TL-2 under MASH.

14. The type "F" steel railing, as shown in the Standard Details of Chapter 40, shall not be used on new bridge plans with a PS&E after 2013. It has not been allowed on the National Highway System (NHS) in the past and was used on non-NHS roadways with a design speed of 45 mph or less. Details in Chapter 40 are for informational purposes only.

15. If a box culvert has a Traffic Railing across the structure, then the railing members shall have provisions for a thrie beam connection at the ends of the structure as shown in the Facilities Development Manual (FDM) SDD 14b20. Railing is not required on box culverts if the culvert is extended to provide an adequate clear zone as defined in FDM 11-15-1. Non-traversable hazards or fixed objects should not be constructed or allowed to remain within the clear zone. When this is not feasible, the use of a Traffic Railing to shield the hazard or obstacle may be warranted. The railing shall be provided only when it is cost effective as defined in FDM 11-45-1.

16. When the structure approach thrie beam is extended across the box culvert; refer to Standard Detail, Box Culvert Details for additional information. The minimum dimension between end of box and face of guard rail provides an acceptable rail deflection to prevent a vehicle wheel from traversing over the end of the box culvert. In almost every case, the timber posts with offset blocks and standard beam guard are used. Type "W" railing may be used for maintenance and box culvert extensions to mitigate the effect of structure modifications.

See the FDM for additional railing application requirements. See FDM 11-45-1 and 11-45-2 for Traffic Barrier, Crash Cushions, and Roadside Barrier Design Guidance. See FDM 11-35-1 Table 1.2 for requirements when barrier wall separation between roadway and sidewalk is necessary.

30.3 General Design Details

1. Epoxy coated bars are required for all concrete parapets, curbs, medians, and sidewalks.

2. Adhesive anchored parapets are allowed at interior Traffic Railing locations only when the adjacent exterior parapet is a crash test approved Traffic Railing per 30.2 (i.e., cast-in-place anchors are used at exterior parapet location). See Standards for Parapet Footing and Lighting Detail for more information.

3. Sign structures, sign trusses, and monotubes shall be placed on top of railings to meet the working width and zone of intrusion dimensions noted in FDM 11-45-2.3.1.1 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. Therefore, slip forming of bridge parapets shall not be allowed.

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). 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. Most snooper trucks have a 10’-0” to 11’-0” vertical allowance to clear fences.

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.

14. When protective screening is warranted, the top of screening should be a minimum of 8-ft above the sidewalk or roadway to discourage people from dropping or throwing objects onto vehicles passing under the structure. For additional protection, the screening may be bent over the sidewalk or roadway. For special aesthetic considerations, a reduced total height of 6’-0” minimum may be considered. However, it should be recognized that the lower screening height provides a reduced level of protection. See (9) for snooper truck vertical access limitations.

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.

30.4 Railing Aesthetics

Railing aesthetics have become a key component to the design and delivery of bridge projects in Wisconsin. WisDOT Regions, local communities and their leaders use rail aesthetics to draw pedestrians to use the walkways on structures. With the increased desire to use, and frequency of use of aesthetics on railings, it has become increasingly important to set policy for railing aesthetics on bridge structures.

Railing aesthetics policies have been around for multiple decades. In the 1989 version of the AASHTO Standard Specifications, generalities were listed for use with designing bridge rails. Statements such as “Use smooth continuous barrier faces on the traffic side” and “Rail ends, posts, and sharp changes in the geometry of the railing shall be avoided to protect traffic from direction collision with the bridge rail ends” were used as policy and engineering judgment was required by each individual designer. This edition of the Standard Specifications aligned with NCHRP Report 350.

Caltrans conducted full-scale crash testing of various textured barriers in 2002. This testing was the first of its kind and produced acceptable railing aesthetics guidelines for single slope barriers for NCHRP Report 350 TL-3 conditions. Some of the allowable aesthetics were: sandblast textures with a maximum relief of 3/8”, geometric patterns inset into the face of the barrier 1” or less and featuring 45° or flatter chamfered or beveled edges, and any pattern or texture with a maximum relief of 2½” located 24” above the base of the barrier. Later in 2002, Harry W. Taylor, the Acting Director of the Office of Safety Design of FHWA, provided a letter to Caltrans stating that their recommendations were acceptable for use on all structure types.

In 2003, WisDOT published a paper titled, “Acceptable Community Sensitive Design Bridge Rails for Low Speed Streets & Highways in Wisconsin”. The goal of this paper was to streamline what railing aesthetics were acceptable for use on structures in Wisconsin. WisDOT policy at that time allowed vertical faced bridge rails in low speed applications to contain aesthetic modifications. For NHS structures, WisDOT allowed various types of texturing and relief based on crash testing and analysis. Ultimately, WisDOT followed many of the same requirements that were deemed acceptable by FHWA based on the Caltrans study in 2002.

NCHRP Report 554 – Aesthetic Concrete Barrier Design – was published in 2006 to (1) assemble a collection of examples of longitudinal traffic barriers exhibiting aesthetic characteristics, (2) develop design guidelines for aesthetic concrete roadway barriers, and (3) develop specific designs for see-through bridge rails. This publication serves as the latest design guide for aesthetic bridge barrier design and all bridge railings on structures in Wisconsin shall comply with the guidance therein.

The aforementioned tests and studies done on aesthetic features will be considered still applicable under MASH barring further tests or studies.

The application of aesthetics on bridge railings on structures in Wisconsin with a design speed exceeding 45 mph shall comply with the following guidance:

1. All Traffic Railings shall meet the crash testing guidelines outlined in 30.1.

2. The top surface of concrete parapets shall be continuous without raised features (pilasters, pedestals, etc.) that potentially serve as snag points for vehicles or blunt ends for impacts. Any raised feature that could serve as a blunt end or snag point shall be placed as follows:

   Minimum of 2’-3” behind the front face toe of the parapet when used with single slope parapets (“32SS”, “36SS”, “42SS”, or “56SS”).

   Minimum of 2’-6” behind the front face toe of the parapet when used with sloped face parapets (“LF” or “HF”).

   Minimum of 2’-0” behind the front face of the parapet when used with vertical face parapets (“A”).

3. Any railing placed on top of a concrete parapet shall be continuous over the full extents of the bridge.

4. Any concrete parapet placed directly on the deck may contain patterns or textures of any shape and length inset into the front face with the exception noted in #5. The maximum pattern or texture recess into the face of the barrier shall be ½”. Note that the typical aesthetic form liner patterns shown on the Standard for Formliner Details are not acceptable for use on the front face of vehicle barriers.

   WisDOT highly recommends the use of smooth front faces of Traffic Railings; especially in high speed applications where the aesthetic features will be negligible to the traveling public. In addition to the increased risk of vehicle snagging, aesthetic treatments on the front face of traffic railings are exposed to vehicle impacts, snowplow scrapes, and exposure to deicing chemicals. Due to these increased risks, future maintenance costs will increase.

5. No patterns with a repeating upward sloping edge or rim in the direction of vehicle traffic shall be permitted.

6. Staining should not be applied to the roadway side face of concrete traffic railings.

The application of aesthetics on bridge railings on structures in Wisconsin with a roadway design speed of 45 mph or less shall comply with the following guidance (see Chapter 4 – Aesthetics for CSD funding implications):

1. All Traffic Railings shall meet the crash testing guidelines outlined in 30.1.

2. The top surface of concrete parapets shall be continuous without raised features (pilasters, pedestals, etc.) that potentially serve as snag points for vehicles or blunt ends for impacts. Any raised feature that could serve as a blunt end or snag point shall be placed a minimum of 1’-0” behind the front face toe of the parapet.

3. Any railing placed on top of a concrete parapet shall be continuous over the full extents of the bridge.

30.6 Protective Screening

Protective screening is a special type of fence constructed on the sides of an overpass to discourage and/or prevent people from dropping or throwing objects onto vehicles passing underneath the structure. Protective screening is generally chain link type fencing attached to steel posts mounted on top of a Traffic Railing (part of a Combination Railing) or on a sidewalk surface (Pedestrian Railing). The top of the protective screening may be bent inward toward the roadway, if mounted on a Traffic Railing and on a raised sidewalk, to prevent objects from being thrown off the overpass structure. The top of the protective screening may also be bent inward toward the sidewalk, if mounted directly to the deck when it is protected by a Traffic Railing between the roadway and a sidewalk at grade. Aesthetics are enhanced by using a colored protective screening which can be coordinated with the color of the structure. See Chapter 30 and Chapter 37 Standard Details for protective screening detail information.

Examples of situations that warrant consideration of protective screening are:

1. Location with a history of, or instances of, objects being dropped or thrown from an existing overpass.

2. All new overpasses if there have been instances of objects being dropped or thrown at other existing overpasses in the area.

3. Overpasses near schools, playgrounds, residential areas or any other locations where the overpass may be used by children who are not accompanied by an adult.

In addition, all pedestrian overpasses should have protective screening on both sides.

Protective screening is not always warranted. An example of when it may not be warranted is on an overpass without sidewalks where pedestrians do not have safe or convenient access to either side because of high traffic volumes and/or the number of traffic lanes that must be crossed.

When protective screening is warranted, the minimum design should require screening on the side of the structure with sidewalk. Designers can call for protective screening on sides without sidewalks if those sides are readily accessible to pedestrians.

Designers should ensure that where protective screening is called for, it does not interfere with sight distances between the overpass and any ramps connecting it with the road below. This is especially important on cloverleaf and partial cloverleaf type interchanges.

Protective screening (or Pedestrian Railing) may be required for particular structures based on the safety requirements of the users on the structure and those below. Roadway designers, bridge designers, and project managers should coordinate this need and relay the information to communities involved when aesthetic details are being formalized.

For highly vulnerable areas, 1” by 1” mesh size should be considered.

Occasionally, access to light poles behind protective screening is required or the screening may need repair. To gain access, attach fence stretchers to the fencing and remove one vertical wire by threading or cutting. To repair, attach fence stretchers and thread a vertical wire in place of the one removed by either reusing the one in place or using a new one.

Fence repair should follow this same process except the damaged fencing would be removed and replaced with new fencing.

The designer should coordinate fence height, fence shape (vertical or bent), and mesh size with the Region and all other applicable agencies.

See 30.3 for additional guidance with regards to snooper truck access, screening height, and straight vs. bent fencing. See 37.3 Protective Screening for additional guidance on pedestrian overpasses. See FDM 11-35-1.8 for additional guidance pertaining to protective screening usage requirements.

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36.1 Design Method

36.1.1 Design Requirements

All new box culverts are to be designed using *AASHTO LRFD Bridge Design Specifications*, hereafter referred to as *AASHTO LRFD*.

36.1.2 Rating Requirements

The current version of *AASHTO Manual for Bridge Evaluation (LRFR)* covers rating of concrete box culverts. Refer to 45.8 for additional guidance on load rating various types of culverts.

36.1.3 Standard Permit Design Check

New structures are also to be checked for strength for the 190 kip Wisconsin Standard Permit Vehicle (Wis-SPV), with a single lane loaded, multiple presence factor equal to 1.0, and a live load factor (γ_{LL}) as shown in Table 45.3-3. See 45.12 for the configuration of the Wis-SPV. The structure should have a minimum capacity to carry a gross vehicle load of 190 kips, while also supporting the future wearing surface (where applicable – future wearing surface loads are only applied to box culverts with no fill). When applicable, this truck will be designated as a Single Trip Permit Vehicle. It will have no escorts restricting the presence of other traffic on the culvert, no lane position restrictions imposed and no restrictions on speed to reduce the dynamic load allowance, IM. The maximum Wisconsin Standard Permit Vehicle load that the structure can resist, calculated including current wearing surface loads, is shown on the plans.

36.2 General

Box culverts are reinforced concrete closed rigid frames which must support vertical earth and truck loads and lateral earth pressure. They may be either single or multi-cell. The most common usage is to carry water under roadways, but they are frequently used for pedestrian or cattle underpasses.

Box culverts used to carry water should consider the following items:

- Hydraulic and other requirements at the site determine the required height and area of the box. Hydraulic design of box culverts is described in Chapter 8.

- Once the required height and area is determined, the selection of a single or multi-cell box is determined entirely from economics. Barrel lengths are computed to the nearest 6 inches. For multi-cell culverts the cell widths are kept equal.

- A minimum vertical opening of 5 feet is desirable for cleaning purposes.

- Fills less than 2-ft supporting traffic require a cast-in-place concrete culvert and epoxy coated bars in the top slab. A minimum of 6-inches of backfill should separate the top slab of culverts and the bottom of the pavement.

Pedestrian underpasses should consider the following items:

- The minimum opening for pedestrian underpasses is 8 feet high by 10 feet wide. However, when considering maintenance and emergency vehicles or bicyclists the minimum opening should be 10 feet high by 12 feet wide. For additional guidance refer to the Wisconsin Bicycle Facility Design Handbook and the FDM.

- The top and sides should be waterproofed with a continuous sheet membrane for the entire length of the culvert.

- The top of the bottom slab should be sloped with a 1% normal crown to minimize moisture collecting on the travel path. Additionally, 0.5% to 1% longitudinal slope for drainage is recommended.

- Flared wings are recommended at openings. For long underpasses, lighting systems (recessed lights and skylights) should be considered, as well. For additional guidance on user’s comfort, safety measures, and lighting refer to the Wisconsin Bicycle Facility Design Handbook.

Cattle underpasses should consider the following items:

- The minimum size for cattle underpasses is 6 feet high by 5 feet wide.

- Consider providing a minimum longitudinal slope of 1%, desirable 3%, to allow for flushing, but not so steep that the stock will slip. Slopes steeper than 5% should be avoided.

- For additional guidance refer to the FDM.

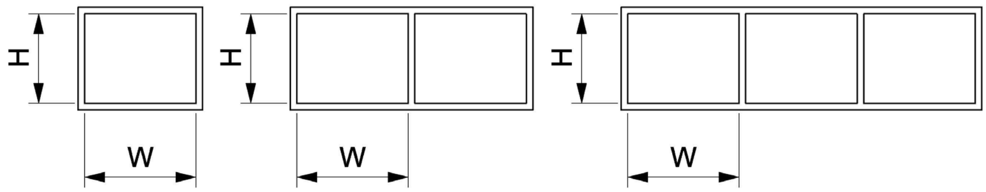

Figure 36.2-1
Typical Cross Sections

36.2.1 Material Properties

The properties of materials used for concrete box culverts are as follows:

f'_c = specified compressive strength of concrete at 28 days, based on cylinder tests

= 3.5 ksi for concrete in box culverts

f_y = 60 ksi, specified minimum yield strength of reinforcement (Grade 60)

E_s = 29,000 ksi, modulus of elasticity of steel reinforcement **LRFD [5.4.3.2]**

E_c = modulus of elasticity of concrete in box **LRFD [C5.4.2.4]**
= $(33{,}000)(K_1)(w_C)^{1.5}(f'_C)^{1/2}$ = 3586 ksi

Where:

K_1 = 1.0

W_C = 0.15 kcf, unit weight of concrete

n = Es / Ec = 8, modular ratio **LRFD [5.6.1]**

36.2.2 Bridge or Culvert

Occasionally, the waterway opening(s) for a highway-stream crossing can be provided for by either culvert(s) or bridge(s). Consider the hydraulics of the highway-stream crossing system in choosing the preferred design from the available alternatives. Estimates of life cycle costs and risks associated with each alternative help indicate which structure to select. Consider construction costs, maintenance costs, and risks of future costs to repair flood damage. Other considerations which may influence structure-type selection are listed in Table 36.2-1.

The consolidation equation is applied to only compressible silts and clays. Sands are of a lower compressibility and no culvert camber is required until the fill exceeds 25 feet. When the fill exceeds 25 feet for sand, a camber of 0.01 feet per foot of fill is used.

36.8.2 Configuration of Camber

The following guides are to be followed when detailing camber.

- It is unnecessary to provide gradual camber. "Brokenback" camber is closer to the actual settlement which occurs.
- Settlement is almost constant from shoulder point to shoulder point. It then reduces to the ends of the culvert at the edge of the fill.
- The ends of the culvert tend to come up if side slopes are steeper than 2½ to 1. With 2 to 1 side slopes camber is increased 10% to compensate for this rise.

36.8.3 Numerical Example of Settlement Computation

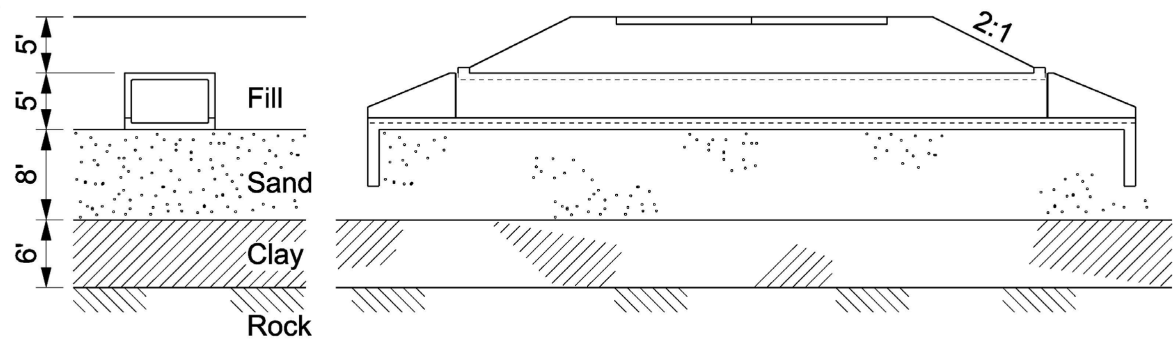

Figure 36.8-1
Soil Strata under Culvert

A box culvert rests on original ground consisting of 8 feet of sand and 6 feet of clay over bedrock. Estimate the settlement of the culvert if 10 feet of fill is placed on the original ground after the culvert is constructed. The in-place moisture content and liquid limit equal 40%. The initial void ratio equals 0.98. The unit weight of the clay is 105 pcf and that of the fill and sand is 110 pcf. There is no water table.

σ'_o = (8 ft)(110 pcf) + (3 ft)(105 pcf) = 1195 psf

$\sigma'_f = \sigma'_o$ + (10 ft)(110 pcf) = 1195 psf + 1100 psf = 2295 psf

C_c = 0.007 (40-10) = 0.21 (approximate value)

$$S_c = \left[\frac{H_c}{1+e_o}\right] c_c \log_{10}\left[\frac{\sigma'_f}{\sigma'_o}\right] = \frac{6\text{ft}}{1+0.98} 0.21 * \log_{10}\left[\frac{2295\text{psf}}{1195\text{psf}}\right] = 0.18\text{ft}$$

36.9 Box Culvert Structural Excavation and Structure Backfill

All excavations for culverts and aprons, unless on bedrock or fill, are to include a 6-inch minimum undercut and backfilled with Backfill Structure Type B. This undercut is for construction purposes and provides a solid base for placing reinforcement and pouring the bottom slab. For fill sections, it is assumed that placed fills provide a solid base and structural backfill is not needed. For cut sections, deeper under cuts and other measures may be warranted to mitigate differential settlement. To ensure greater base stability, breaker run or similar material, is typically provided over geotextile fabric. For precast concrete box culverts, a 6-inch minimum bedding material with 100% passing the 2-inch sieve shall be provided for uniform bearing support. If precast box culverts require Breaker Run material for base stability, a separation barrier should be provided between the bedding and the Breaker Run material.

All volume excavated and not occupied by the new structure should be backfilled with Backfill Structure Type B for the full length of the box culvert, including the apron.

See Standard Detail 9.02 – Structure Backfill Limits and Notes 2– for typical pay limits and plan notes.

36.11 Plan Detailing Issues

36.11.1 Weep Holes

Investigate the need for weep holes for culverts in cohesive soils. These holes are to relieve the hydrostatic pressure on the sides of the culverts. Where used, place the weep holes 1 foot above normal water elevation but a minimum of 1 foot above the lower sidewall construction joint. Do not place weep holes closer than 1 foot from the bottom of the top slab.

36.11.2 Cutoff Walls

Where dewatering the cutoff wall in sandy terrain is a problem, the concrete may be poured in the water. Place a note on the plans allowing concrete for the cutoff wall to be placed in the water.

36.11.3 Name Plate

Designate a location on the wingwall for placement of the name plate. Locate the name plate on the roadway side of the first right wing traveling in the highway cardinal directions of North or East.

36.11.4 Plans Policy

If cast-in-place concrete box sections or aprons are used, full plans shall be provided and sealed by a professional engineer. The plans shall be in accordance with the *Bridge Manual* and Standards.

If precast concrete box sections are allowed in lieu of cast-in-place concrete, a noted allowance shall be provided on the plans. Precast details are not required for box sections following ASTM Specification C1577. The design and fabrication shall be in accordance with ASTM Specification C1577, AASHTO LRFD Specifications, and the Bridge Manual.

If precast only concrete box sections are justified, precast details are required for box sections following ASTM Specification C1577. The design and fabrication shall be in accordance with ASTM Specification C1577, AASHTO LRFD Specifications, and the Bridge Manual.

If precast concrete apron elements are allowed, a noted allowance shall be provided on the plans and precast details shall be provided in accordance with the *Bridge Manual* and Standards. The design may deviate (e.g. use a precast apron floor) from the precast alternatives shown in the Standards provided the engineer submits design calculations, sealed by a professional engineer, to the Bureau of Structures for acceptance. The design and fabrication shall be in accordance with AASHTO LRFD Specifications and the Bridge Manual.

If the contractor selects a precast alternative, the contractor is to submit shop drawings, sealed by a professional engineer, to the Bureau of Structures for acceptance. If precast concrete elements (e.g. apron wingwalls) are prohibited by the designer, the plans shall be noted accordingly.

36.11.5 Rubberized Membrane Waterproofing

When required by the Standard Details, place the bid item "Rubberized Membrane Waterproofing" on the final plans. The quantity is given square yards.

36.12 Precast Four-Sided Box Culverts

Typically, precast concrete box culverts can reduce construction time, but may also cost more than cast-in-place concrete construction. As such, it is often difficult to determine if a contractor will choose to use precast or cast-in-place sections. To provide greater flexibility, projects can provide options (alternatives) for the contractor to determine if precast would be beneficial based on the project's needs.

In general, there are two options for preparing concrete box culvert plans. The most common and recommended option is to provide a complete cast-in-place concrete design with a noted allowance for the contractor to substitute the cast-in-place design with precast box sections in accordance with ASTM C1577. This option provides project flexibility while maintaining historically lower cast-in-place concrete costs. The designer shall determine if a noted precast allowance is appropriate on a project-by-project basis. In some cases, the precast option may not be suitable and should be noted accordingly on the plans. The following are several conditions where a noted allowance for precast may not be suitable for a project:

- Structure openings not covered by ASTM Specification C1577, which will require a separate analysis. ASTM C1577 provides standard designs for single cell sections with openings ranging from 3-ft span x 2-ft rise to 12-ft span x 12-ft rise. Special designed sections, not covered by ASTM C1577, are subject to prior-approval by the Bureau of Structures.

- Structure skew is greater than 30 degrees and the depth of cover is less than 5 feet. This condition is beyond the design tables shown in ASTM C1577 and requires a separate analysis.

- Depth of cover less than 2 ft while supporting traffic loads. Cast-in-place sections are required due to performance concerns at the top slab and joint locations.

- Pedestrian underpasses - Cast-in-place sections are required for improved serviceability.

- Unique hydraulic conditions or other factors may also warrant not allowing precast sections, such as differential settlement concerns.

A precast concrete only plan delivery method may be considered when cast-in-place concrete usage is highly unlikely. This option would simplify plan preparation and may provide design savings. Use of precast only culverts, that are assigned a structure number, are subject to prior-approval by the Bureau of Structures.

If precast concrete box sections are allowed, the designer shall also determine if precast aprons should be allowed as well. Use of precast aprons may not be as beneficial as concrete box sections since these elements are located beyond the construction staging limits and may not require an accelerated schedule.

Refer to 36.11.4 for additional information on plan detail requirements.

36.13 Other Buried Structures

The following section provides general guidance on cross-drain alternatives to concrete box culverts.

36.13.1 General

Typical alternatives to four-sided (box) concrete structures include three-sided (bottomless) concrete structures and metal buried structures. These structures are available in a variety of shapes, sizes, and material types. In general, three-sided structures may be cost prohibitive when deep foundations are required.

Concrete buried structures are rigid structures that can be constructed using cast-in-place or precast concrete. These structures obtain strength through reinforced concrete sections that have proven to be durable and long-lasting. Refer to 36.13.2 for additional information on three-sided concrete structures.

Metal buried structures are typically constructed with factory assembled corrugated sections or field assembled structural plates. Commonly used shapes include pipes and pipe-arches consisting of steel or aluminum alloy. These flexible structures obtain strength through soil-structure interactions that allow for the use of thin-walled sections. Some advantages of metal buried structures include; increased speed of installation, potential initial cost savings, and the variety of available shapes. Some disadvantages include their susceptibility to damage and/or degradation and performance being dependent on the quality of installation. Refer to 36.13.3 and FDM 13-1 for additional information on metal buried structures.

Buried structures assigned a structure number shall be coordinated with the Bureau of Structures and follow the policies and procedures as stated in the Bridge manual and FDM 13-1. Refer to 2.5 for information on assigning structure numbers.

Refer to AASHTO LRFD Section 12 – Buried Structures and Tunnel Liners for additional information.

36.13.2 Three-Sided Concrete Structures

Three-sided box culvert structures are divided into two categories: cast-in-place three sided structures and precast three-sided structures. These structures shall follow the criteria outlined below.

36.13.2.1 Cast-In-Place Three-Sided Structures

To be developed

36.13.2.2 Precast Three-Sided Structures

Three-sided precast concrete structures offer a cost effective, convenient solution for a variety of bridge needs. The selection of whether a structure over a waterway should be a culvert, a three-sided precast concrete structure or a bridge is heavily influenced by the hydraulic

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40.1 General

New bridges are designed for a minimally expected life of 75 years. Preliminary design considerations are site conditions, structure type, geometrics, and safety. Refer to Bridge Manual Chapters 9 and 17 for Materials and Superstructure considerations, respectively. Comprehensive specifications and controlled construction inspection are paramount to obtaining high quality structures. Case history studies show that adequately consolidated and properly cured concrete with low water-cement ratios and good air void systems have lower absorption rates and provide greater resistance to scaling and chloride penetration under heavy traffic and exposure to de-icing chemicals. Applying protective surface treatments to new decks improves their resistance to first year applications of de-icing chemicals.

Most interstate and freeway structures are not subject to normal conditions and traffic volumes. Under normal environmental conditions and traffic volumes, original bridge decks have an expected life of 40 years. Deck deterioration is related to the deck environment which is usually more severe than for any of the other bridge elements. Decks are subjected to the direct effects of weather, the application of chemicals and/or abrasives, and the impact of vehicular traffic. For unprotected bar steel, de-icing chemicals are the primary cause of accelerated bridge deck deterioration. Chlorides cause the steel to corrode and the corrosion expansion causes concrete to crack along the plane of the top steel. Traffic breaks up the delaminated concrete leaving potholes on the deck surfaces. In general, deck rehabilitation on Wisconsin bridges has occurred after 15 to 22 years of service due to abnormally high traffic volumes and severe environment.

Full depth transverse floor cracks and longitudinal construction joints leak salt water on the girders below causing deterioration and over time, section loss.

Leaking expansion joints allow salt water seepage which causes deterioration of girder ends and steel bearings located under them. Also, concrete bridge seats will be affected in time. Concrete bridge seats should be finished flat, and sealed with a penetrating epoxy coating.

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

removed to at least the original deck surface. Additional surface milling may not be practical if the previous overlay included a milling operation.

40.5.1.3 Polyester Polymer Concrete Overlay

A polyester polymer concrete (PPC) is expected to extend the service life of a bridge deck for 20 to 30 years. This system is a mixture of aggregate, polyester polymer resin, and initiator; which can be placed as a deck overlay using conventional concrete mixing and placement equipment, albeit most likely dedicated to PPC usage. The main advantages of a PPC overlay is that it is impermeable and causes minimal traffic disruptions due to its quick cure time. High costs and lack of performance data are the main disadvantages.

Prior to the placement of the PPC overlay, a high molecular weight methacrylate (HMWM) binder is placed on the prepared deck. This bonds the overlay to the deck, and it also serves to seal existing cracks in the deck. When the existing concrete is in good condition, PPC is effective at mitigating chloride penetration due to its impermeability. In some situations, PPC has exhibited reflective cracking from the deck below. Cracks should be sealed with methacrylate sealer as recommended by the particular PPC manufacturer.

The total thickness of a PPC overlay is typically 3/4” to 1”. While thicker overlays are possible, they are usually cost prohibitive. PPC can be placed at 3/4” thick as opposed to a typical 1 1/2” thick concrete overlay. This may help in situations where bridge ratings and/or profile adjustments are of concern but should not be the primary reason for applying PPC.

Since most applications recommend a 1-inch or less overlay, PPC overlays are considered a thin polymer overlay and have similar requirements and restrictions. PPC overlays should be limited to decks in good condition that require shorter traffic disruptions for sites with high traffic volumes and lane closure restrictions. PPC is a durable product and has a relatively fast curing time (2 to 4 hours), but also has a higher cost as compared to a concrete overlay. PPC overlays should be used based on the following restrictions:

- Deck wearing surface distress should not exceed 5% of the total deck area.

- Decks should have a NBI rating of 6 or greater and be less than 20 years old. Older decks may be considered when the existing deck has been protected by a thin polymer overlay or when chloride testing indicates acceptable chloride levels at the reinforcement. Chloride contents at the reinforcement should not exceed 2 lbs/CY for decks with epoxy coated reinforcement. PPC overlays are not recommended on decks with uncoated top mat reinforcement. Decks exposed to chlorides, exceeding 10 years, should consider a ¾-inch minimum scarification to remove chlorides.

- PPC overlays should not be placed on concrete decks or Portland cement concrete patches less than 28 days, unless approved otherwise. Patch and crack repairs shall be compatible with the overlay material.

- PPC shall not be used for structural repairs due to costs and performance concerns.

- PPC should not be used unless lower cost preservation treatments (e.g. thin polymer overlays) have proven ineffective. If a bridge deck has a TPO, chloride ion testing will be performed near the end of the TPO life to determine eligibility of either a TPO reapplication or a PPC overlay (if the structure also meets the condition and ADT criteria). If the average chloride concentration at 1" depth is less than 1 lb/cy, the existing TPO is considered an effective preservation treatment. The TPO should be replaced with another TPO. If the average chloride concentration at 1" depth is greater than 1 lb/cy, a PPC overlay should be considered (including a ¾"-1" milling of the deck surface to remove chlorides).

Note: PPC overlays are expensive and new to WisDOT. As a result, use of PPC overlays should be limited to preservation projects that meet the requirements outlined in Figure 40.5-2 **or as approved by the Bureau of Structures.**

Other factors which may affect BOS approval of PPC overlays include:

- Proximity to other high ADT roadways (i.e., service ramp)
- Backbone/Interstate or high priority structure
- Preservation of slab structure
- Presence of active cracking (not recommended when active cracking is present)
- Enhanced friction (not to be used as a ride correction or as a high-friction improvement)

40.5.1.4 Polymer Modified Asphaltic Overlay

A polymer modified asphaltic (PMA) overlay is expected to extend the service life of a bridge deck for 10 to 15 years. This system is a mixture of aggregate, asphalt content, and a thermoplastic polymer modifier additive, which can easily be placed as a deck overlay using conventional asphalt paving equipment. The thickness of the overlay is 2-inches minimum and can accommodate profile and cross-slope differences.

The added polymer allows for the overlay to resist water and chloride infiltration. Proper mix control and placement procedures are critical in achieving this protection. Core tests have shown the permeability of this product is dependent on the aggregate. As a result, limestone aggregates should not be used.

PMA overlays can be used on more flexible structures (e.g. timber decks or timber slabs) and to minimize traffic disruptions.

Designers should contact the region to determine if a PMA overlay is a viable solution for the project. In some areas, product availability or maintaining an acceptable temperature may be problematic.

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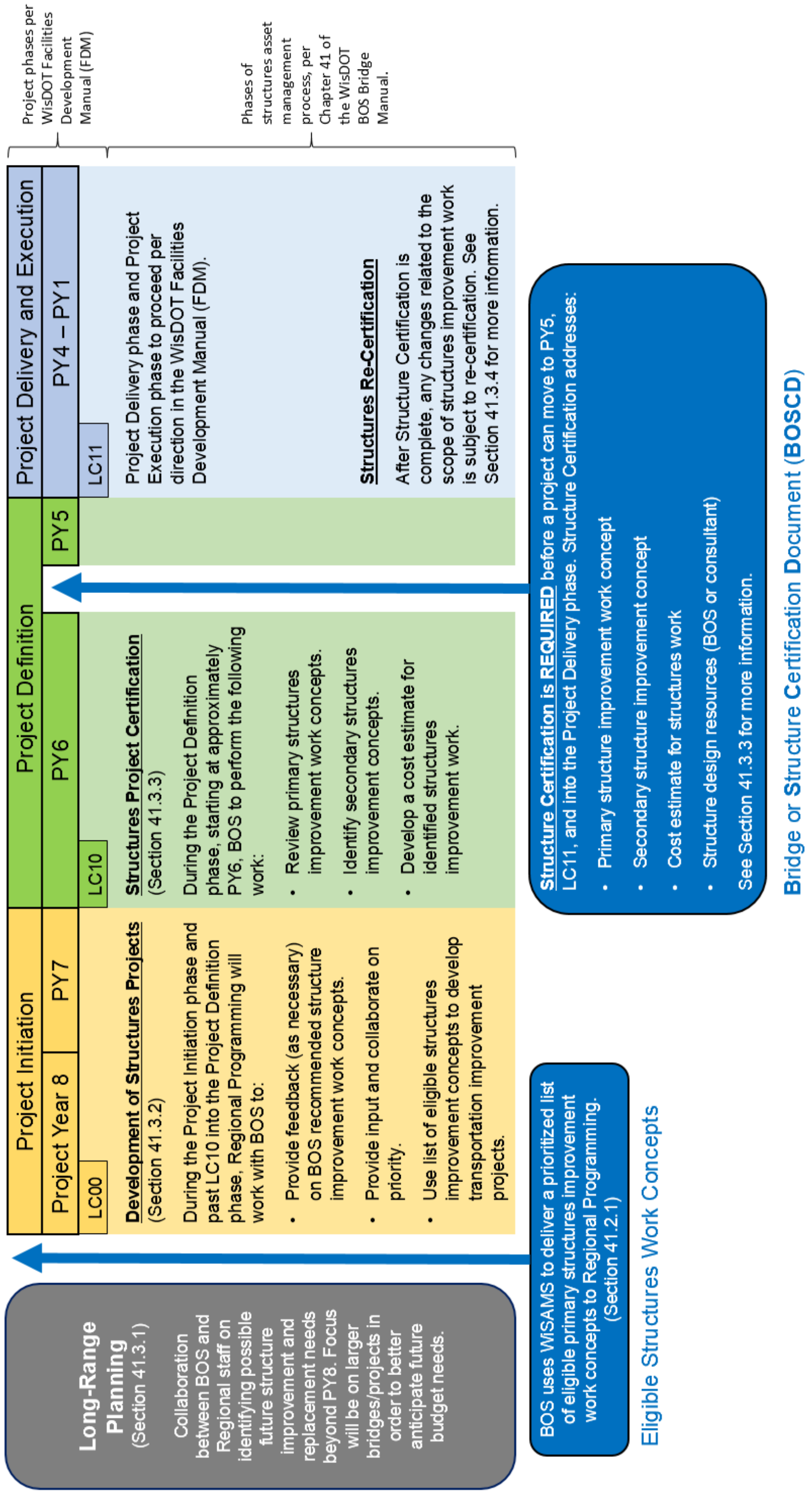

Figure 41.3-1
WisDOT Structures Asset Management – Structures Project Development

41.3.1 Long-Range Planning

Long-range planning refers to planning work done for projects with a target year beyond Program Year 8. Long-range planning serves several purposes, including examples such as:

- Coordinates improvement projects that are close in proximity to each other to minimize inconvenience for the travelling public.

- Project future improvement needs to large and/or complex bridges. Work of this nature may have a large impact in terms of budget and required design time.

- Provide information on future structure needs to coordinate with the long-term Division and Department vision for targeted corridors or areas.

- Provide a network-wide projection of future needs to be used when considering future transportation funding levels.

Projection of long-range structure improvement needs are based on WiSAMS output. BOS and Region collaboration on long-range planning occurs on an as-needed basis.

41.3.2 Development of Projects with Structures Work (PY8-PY7, Life Cycle 00-10)

The process of developing structure projects initiates with the BOS. Using WiSAMS (described above in 41.2.1) and review by BOS asset management engineers, BOS develops a list of eligible structures work concepts for the target year – Program Year 8 (PY8). The work is based on established BOS and Department policies for structures asset management, as described in this chapter and Chapter 42 – Bridge Preservation. The list of eligible structures work concepts is also prioritized. BOS will deliver these work concepts to the Regions twice annually, in February and August, utilizing the Structures Certification Tool (SCT).

41.3.2.1 Optional Work Concept Review

When eligible structure work concepts are published in the SCT, BOS inspection and maintenance personnel have the opportunity, though not required, to review the eligible structure work concepts before they are selected for improvement projects. The focus of this review is on the primary work concepts, though some secondary work concepts may also be identified at this stage.

BOS inspection and maintenance review should be focused on identifying perceived gross mismatches in scope and/or timing, and highlighting structure work concepts not identified by WiSAMS. Final decisions on scope and timing must be based on data and/or documentation. A majority of the time, this will be WiSAMS, but it can also be supplemented by other information, such as construction history, supplemental inspection data, IR data, or any other information pertinent to the programming decision. Final scope and timing decisions for structures work will be made by BOS asset management engineers, with strong consideration of BOS inspection and maintenance personnel input.

41.3.2.2 Priority Review

BOS provides a prioritized list of eligible structures work. Priority is determined using a priority index (PI); an algorithm developed by BOS. The algorithm considers data such as ADT, functional class, etc. This is intended to assist the regions as they program projects.

The Region may see fit to adjust the prioritized list based on regional system and operational factors.

41.3.2.3 Creating Improvement Projects with Structures Work Concepts

The next step in the programming process is for Regional Programming to develop structures improvement projects based on the list of individual structures work concepts. Projects may combine structures work as appropriate, but also consider pavement needs, safety needs, operational needs, etc.

There may be non-structural rationale for deviations from BOS-recommended scope and/or timing. Common reasons include, but are not limited to:

- Coordination with other improvement work (pavements, safety, operations, etc.)
- Traffic control costs
- User delay

If reasons such as those noted above are used to justify deviations from BOS-recommended scope and/or timing, a cost-benefit analysis should be performed to support the decision. More information on cost-benefit analysis and structures programming policy can be found in 41.6.6.

During this phase as projects are developed and up until the Structures Project Certification Phase (See 41.3.3), BOS asset management engineers will evaluate proposed projects on a regular basis to ensure that programmed structures work is eligible in terms of both scope and timing. Projects that contain only eligible structures work concepts or have appropriate justification for any deviations are considered *pre-certified*.

<u>Only eligible projects or projects with appropriate justification will be considered for funding.</u>

41.3.3 Structures Project Certification Phase (PY6-PY5, Life Cycle 10/11)

Structures project certification refers to the work required to produce the Bridge or Structure Certification Document (BOSCD). The components of the BOSCD are outlined in 41.3.3.6 below.

<u>WisDOT policy item:</u>
Any improvement project with state-owned B-Structure work (primary or secondary work concepts) requires certification.

41.3.3.1 BOS Structures Certification Liaison

BOS will designate a certification liaison for every structures improvement project, regardless of whether the project is designed by BOS or a consultant. The certification liaison will perform all of the work necessary for structures certification. A certification liaison will remain with each structures project (BOS-designed or consultant-designed) through the letting of that project, though the actual person assigned to a project may change over the lifecycle of that project.

41.3.3.2 Review of Primary Structures Work Concepts

Structures certification serves as the final review and approval for the scope and timing of the primary structures work concept. Regional planning engineers should only be selecting eligible structures work (scope and timing) for inclusion in transportation improvement projects. Additionally, BOS asset management engineers will evaluate projects on a regular basis (see 41.3.2) to ensure eligibility. With this process in place, the certification liaison will collaborate with BOS asset management engineers and Regional programming engineers (as necessary) to confirm scope and timing for primary structures work concepts.

41.3.3.3 Development of Secondary Structures Work Concepts

A key portion of the BOSCD is the early identification of secondary structures improvement work. Some examples of secondary work include, but are not limited to:

- Bearing rehabilitation or replacement
- Parapet or railing repairs
- Backwall or wingwall repairs
- Identification of specific substructure repairs
- Scour mitigation

Some items such as those above may have already been identified during the scoping of the primary structures work concepts. The certification liaison will review the existing inspection reports on file and consult the appropriate BOS inspection and maintenance personnel to identify any and all eligible secondary structures work concepts.

41.3.3.4 Development of the Structures Cost Estimate

A high-level cost estimate will have been developed as a part of the primary structures work concept. This estimate is for structures work only; costs for traffic control and mobilization are not included. The certification liaison will refine that estimate, taking into account the identified secondary structures improvement work. This estimate is not intended to be a final structures construction cost estimate, but is a refinement of the unit cost estimate previously developed.

41.3.3.5 Determination of Design Resourcing

As part of the structure's certification process, BOS will determine design resourcing and estimate the level of effort (in staff-hours) for the structures work. If BOS chooses to decline structures design for a given project, regional PDS staff should work with BOS consultant review supervisor to ensure selection of an appropriate consultant engineer for the project.

41.3.3.6 Bridge or Structure Certification Document (BOSCD)

The BOSCD includes information on all the items noted above, in addition to other key information identified by Region personnel. Additional project information and decision documentation can be found in the SCT.

41.3.4 Project Delivery and Execution Phase (PY4-Construction, Life Cycle 12+)

41.3.4.1 Structures Re-Certification

Any and all changes related to structures improvement work affecting items approved as part of the structures project certification shall be reviewed and approved by the certification liaison. This includes, but is not limited to, any of the following items:

- Scope (primary or secondary)
- Structures construction cost estimate
- PS&E or let date
- Advanceable date
- Structures design resourcing

The certification liaison for the project should be notified of any changes as soon as reasonably possible to approve/re-certify the project in a timely manner and not delay project schedule.

41.4 Structures Programming Process (Local System)

In general, local entities that own transportation structures may expend resources to preserve, rehabilitate, and replace structures at the owner's discretion. The state does require minimum information regarding all structures utilized for public transportation, and should be informed of structure work affecting the performance and/or capacity of the structure.

Local structure work may also be funded through the Local Bridge Assistance Program (Local Program). To support this program, BOS provides a prioritized eligibility list of bridge work concepts for the Division of Transportation Investment Management (DTIM), which is then posted publicly for the local owners. Local owners use the eligibility list to select projects for submission to the local program, and DTIM programs structure work on a biannual basis. Submission of eligible bridge work does not guarantee an entitlement of funds. According to Trans 213.03(4)(e), applications must both be approved and prioritized before determining entitlement of funds.

Not all bridges will have a work concept listed in the eligible bridge list. If an applicant believes work is necessary for a bridge that does not have a proposed work concept, or if the applicant believes a different work concept than the proposed work concept is more appropriate, the applicant can submit an alternate work concept. This will require an engineering report attached to the application for funding which describes the work concept proposed to be done, justification for the new work concept, and a life cycle cost analysis of different alternatives. Additional program requirements may apply.

More information about the Local Bridge Assistance Program can be found at the following link:
https://wisconsindot.gov/Pages/doing-bus/local-gov/astnce-pgms/highway/localbridge.aspx

41.4.1 Eligible Project Scopes

The structure work described in this section applies to eligible project scopes submitted in applications to the Local Bridge Assistance Program. These structure work scopes can be submitted as an application for Preservation funding, Rehabilitation funding, or Reconstruction funding. The descriptions below are intended to be general descriptions of the primary work being proposed. Project scope may include other secondary (lesser) work as needed. Detailed information and guidance regarding the specific project scopes listed below may be found extensively throughout the Bridge Manual.

Preservation Project Scopes may include:

Thin Polymer Overlay – A polymer resin with broadcast aggregate, applied in two separate layers. Total thickness is typically 1/4" to 3/8" thick.

Polyester Polymer Concrete Overlay – A pre-mixed polymer and aggregate concrete. Total thickness is typically 3/4" to 1" thick.

Hot Mix Asphalt Overlay with Membrane – An asphaltic concrete placed on top of a waterproof membrane. Total thickness is typically 2" to 3" thick. (The surface of this overlay may be resurfaced or repaired with standard asphaltic concrete while maintaining the membrane below.)

Polymer Modified Asphalt Overlay – An asphaltic concrete with an internal membrane utilizing polymer additives. Minimum thickness is typically 1.5” thick.

Rehabilitation Project Scopes may include:

Concrete Overlay – A Portland cement concrete. Minimum thickness is typically 1.5” thick.

Deck Replacement – Removal and replacement of the existing concrete deck.

Paint – Full painting of steel superstructure (beams, girders, bracing, etc.).

Superstructure Replacement – Removal and replacement of the existing superstructure (and deck as applicable).

Reconstruction Project Scopes may include:

Replace Structure – Full removal and replacement of the existing structure.

41.5 Structures Asset Management Roles and Responsibilities

41.5.1 Bureau of Structures (BOS)

BOS has three sections, each of which contribute to the structures asset management process, either directly or indirectly.

BOS Design Section

- Resource the design (including hydraulic considerations) of structures improvement projects or providing oversight for consultant-designed projects.
- Provide resources (certification liaison) for the structures project certification (See 41.3.3).

BOS Maintenance Section

- Provide oversight for the WisDOT structures inspection program, working to ensure and improve the quality and accuracy of condition data.
- Provide detailed structures condition data (via inspection reports) that fully and accurately depict the current state of each individual structure.
- Review eligible structures work concepts within SCT, providing additional condition information to support recommend adjustments as deemed necessary.
- Collaborate with BOS certification liaison in the structures certification process, specifically in the scoping of primary and secondary structures work concepts (See 41.3.3.3).
- Perform or coordinate some preventative maintenance work; deck washing, deck sealing, crack sealing, etc. See Chapter 42 – Bridge Preservation for more information.

BOS Development Section

- Manage and maintain the Highway Structures Information System (HSIS), an on-line database for collecting structures inventory and condition data.
- Develop, maintain, and refine Chapter 42 – Bridge Preservation. Policy documented in this chapter is the basis for WisDOT structures asset management.
- Develop and maintain WiSAMS, the software application that uses inspection and inventory data to produce recommendations for future structure improvement projects.
- Using WiSAMS (including priority and budget features), develop draft recommendations for the program-level scope of recommended structures work for the 8-year structures improvement program.
- Collaborate with Regional personnel to develop structures projects for the 8-year structures improvement program.
- Review and pre-certify structures projects that are introduced to the 8-year structures improvement program. See 41.3.2.3.
- Develop and maintain a program effectiveness measure to assess progress toward achieving program goals.

41.5.2 WisDOT Regions

WisDOT divides the state into five regions; Northwest, North Central, Northeast, Southeast, and Southwest. See Figure 2.1-3. Each Region has the responsibilities outlined below for the structures in their designated territory.

Regional Planning and Scoping Units

- Review structures work concepts provided by BOS and coordinate with other stakeholders (pavements, operations, safety, etc.) to recommend adjustments as deemed necessary.
- Collaborate with BOS to develop structures improvement projects that incorporate identified structure needs, coordinating as appropriate to address other need areas (pavement, safety, etc.).
- Collaborate with BOS in the structures certification process (See 41.3.3).

Regional Project Development Sections (PDS)

- Participate in the structures certification process, as necessary (See 41.3.3).
- Coordinate with BOS on structures project re-certification, as necessary. (See 41.3.4.1.)
- Guide structures improvement projects from project certification through construction, working to ensure that the project is constructed per plans and specifications.

41.5.3 Division of Transportation Investment Management (DTIM)

DTIM is responsible for the financial component of structures asset management, determining the allocation of funds for structures improvement projects.

Bureau of State Highway Programs (BSHP)

- Collaborate with BOS to assess structures needs as they relate to the allocation of available funds to the various WisDOT funding programs.
- Determine the specific allocation of available funding for each of the WisDOT funding programs.
- Provide direct oversight and prioritization for the state-wide Backbone funding program.
- Provide financial analysis expertise and tools, such as Life Cycle Cost Analysis (LCCA) guidance.

Bureau of Transit, Local Roads, Railroads & Harbors (BTLRRH)

- Provide direct oversight and programming for the Local Bridge program, utilizing the list of eligible structure work concepts provided by BOS.

41.6 Programming Policy for Structures Improvement Projects

Structures improvement needs are identified by BOS as detailed 41.2 above. As Regional personnel work to develop projects to address these structures needs, other factors may contribute to the final project scope and timing. The policy items noted below provide direction on how some of these project factors shall be considered as they relate to the scope of structures improvement work.

41.6.1 Bridge Age

WisDOT policy item:

Bridge age shall not be a primary driver for the initiation of structures improvement work.

For a given bridge, there is correlation between the condition of the bridge and its age. However, condition (not age) shall be the primary driver for structures improvement work. The focus of evaluation should be on how the structure is currently performing, regardless of structure age.

41.6.2 Bridge Ratings

WisDOT policy item:

Unless specifically approved by BOS, inventory rating, operating rating, or the presence of a load posting shall not be the primary driver for the initiation of structures improvement work.

If a structures improvement project has been reviewed and approved by BOS (see 41.3.3), it may be appropriate to include work to improve load ratings or remove a load posting. It is strongly recommended to perform rating analysis early for a rehabilitation project to identify potential strengthening needs. Consult with the BOS Rating Unit before expanding structures scope to include strengthening.

41.6.3 Vertical Clearance

WisDOT policy item:

Vertical clearance shall not be the primary driver for the initiation of structures improvement work.

Various impact mitigation techniques shall be evaluated for bridges with a history of impacts before scoping an improvement project to include addressing substandard vertical clearance.

If deck replacement, superstructure replacement, or structure replacement are identified as the appropriate treatment and vertical clearance is substandard, the project team should investigate the additional cost of creating more vertical clearance.

Region and BOS concurrence is required to up-scope a project for vertical clearance issues.

41.6.4 Hydraulics

<u>WisDOT policy item:</u>
In the case of structures with flooding history or concerns, improvement work shall not be initiated unless mitigation (detours) are not possible. If mitigation is not possible, consult BOS Hydraulics Unit for direction.

In most cases, traffic can be adequately detoured around flooded structures until such time as waters recede.

41.6.5 Freight Considerations

<u>WisDOT policy item:</u>
Freight needs shall not drive the initiation of a structures improvement project.

As related to structures, freight needs are primarily capacity (load ratings and/or load postings) and clearance (vertical and horizontal).

41.6.6 Cost Benefit Analysis

When considering different options for structures improvement work, a cost-benefit analysis should be performed. The analysis should be performed by Regional programming staff using analysis tools approved by the DTSD Administrator’s Office. Direction on select input data to be used for cost-benefit analysis is detailed below.

41.6.6.1 Treatment Schedule

When performing cost-benefit analysis, the following shall be used as the idealized treatment schedule for a new bridge. **The treatment schedules below are only for use in cost-benefit analysis and <u>are not intended to be used for programming purposes</u>.**

Prestressed Girder Superstructure

Primary Work Concept	Secondary Work Concept	Structure Year
New Construction	---	Year 0
Reseal Deck	---	Year 4
Reseal Deck	---	Year 8
Thin Polymer Overlay	---	Year 12
Thin Polymer Overlay	---	Year 22
Concrete Overlay and New Joints	• Substructure repair • Superstructure repair	Year 47
Deck Replacement	• Substructure repair • Superstructure repair	Year 67
Reseal Deck	---	Year 71
Reseal Deck	---	Year 75
Thin Polymer Overlay	---	Year 79
Thin Polymer Overlay	• Substructure repair • Superstructure repair	Year 89
Bridge Replacement	---	Year 100

Steel Girder Superstructure

Primary Work Concept	Secondary Work Concept	Structure Year
New Construction	---	Year 0
Reseal Deck	---	Year 4
Reseal Deck	---	Year 8
Thin Polymer Overlay	---	Year 12
Thin Polymer Overlay	---	Year 22
Concrete Overlay and New Joints	• Spot/zone painting • Substructure repair • Superstructure repair	Year 47
Deck Replacement	• Complete painting • Substructure repair • Superstructure repair	Year 67
Reseal Deck	---	Year 71
Reseal Deck	---	Year 75
Thin Polymer Overlay	---	Year 79
Thin Polymer Overlay	• Spot/zone painting • Substructure repair • Superstructure repair	Year 89
Bridge Replacement	---	Year 100

Concrete Slab Superstructure

Primary Work Concept	Secondary Work Concept	Structure Year
New Construction	---	Year 0
Reseal Slab	---	Year 4
Reseal Slab	---	Year 8
Thin Polymer Overlay	---	Year 12
Thin Polymer Overlay	---	Year 22
Concrete Overlay and New Joints	• Substructure repair • Superstructure repair	Year 47
Concrete Overlay and New Joints	• Substructure repair • Superstructure repair	Year 67
Bridge Replacement	---	Year 87

For all other superstructure types or in-service structures, consult BOS Bridge Management Unit for direction.

41.6.6.2 Discount Rate

WisDOT policy item:

A discount rate of 5% shall be used for cost-benefit analysis.

This value was determined based on analysis conducted by DTIM and is Department policy.

41.6.7 User Delay

WisDOT policy item:

For the purposes of cost-benefit analysis, user delay shall be addressed per direction in the WisDOT Facilities Development Manual (FDM).

User delay can have a dramatic impact on the results of a cost-benefit analysis and must be considered based on Department policy.

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

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. These bridges have the potential 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.

42.3 Bridge Preservation Actions

This chapter focuses on bridge preservation actions that relate to preventive maintenance and element rehabilitation. Cyclical and condition-based activities are subsets of preventative maintenance as shown below in Figure 42.3-1. Descriptions of these preservation actions can be found in 42.7.

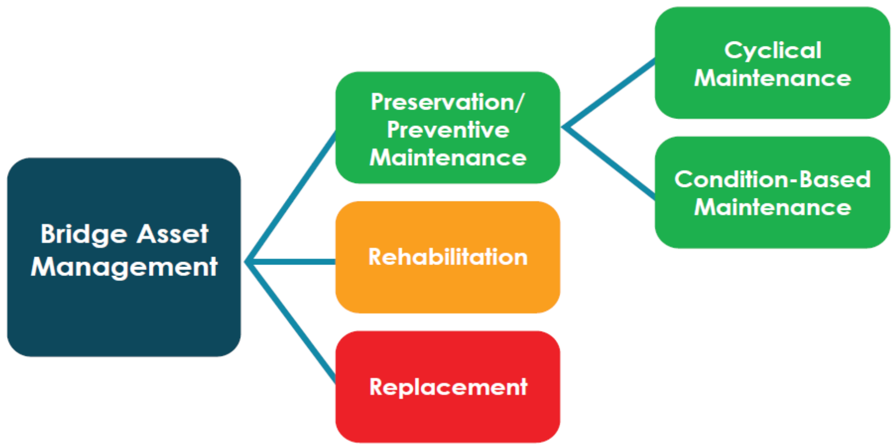

Figure 42.3-1
Asset Management and Preservation Actions

Major rehabilitation, bridge replacement, improvement, and new bridge construction projects are addressed by other WisDOT Bridge Programs.

42.4 Bridge Preservation Goals, Strategies and Performance Measures

This chapter outlines preservation goals, strategies and performance measures to track progress. Maintaining safe and dependable operations is a high priority for the department. **The Department has the goal to maintain 95% of the state-owned bridges in fair or better condition** (NBI component ratings 5 or higher). To achieve this goal, the department employs strategies that include condition and cyclical treatments.

42.4.1 Condition Based Strategies

Condition based preventive maintenance activities are performed on bridge elements as needed and identified through the bridge inspection process. To achieve the goal of maintaining 95% of the state system bridge inventory in fair or better condition (per NBI condition ratings), the department must maintain the bridge components directly associated with this goal. These include:

- Bridge decks in fair or better condition.
- Bridge superstructures in fair or better condition.
- Bridge substructures in fair or better condition.

The department must also maintain key bridge elements and protective systems that preserve the larger bridge components directly associated with the overall bridge condition goal. These elements include:

- Minimize leaking of deck joints.
- Minimize corrosion of steel superstructure elements.
- Minimize restricted movement/function of bearing elements.

42.4.2 Cyclical Based Strategies

Cyclical based activities are performed on a pre-determined interval and aim to preserve existing bridge element or component conditions. These types of activities may not improve the condition of the bridge element or component directly, but will delay their deterioration. Examples of cyclical activities include:

- Deck sweeping
- Deck and Superstructure washing
- Deck sealing

42.4.3 Performance Measures and Objectives

Performance measures in this chapter are consistent with the objectives of the program and reflect the experience and input of the WisDOT Regional Bridge Maintenance Staff as well as consideration of other DOT's insight and experience.

Table 42.4-1 lists the measures and objectives for preservation program performance:

Objective	Inventory Performance Measure	Component/ Element Reference	Condition Performance Measure
Maintain bridges in good or fair condition	95% of bridges	B.C.01-04	NBI component rating 5 or higher
Maintain bridge decks in good or fair condition	95% of bridge decks	B.C.01	Deck NBI component rating 5 or higher
Maintain effective expansion joints that do not leak	85% of bridges with strip seal joints	Defect 2310 of Element 300	90% of strip seal expansion joints in condition state 2 or better
Maintain coated steel surfaces of superstructure elements	90% of bridges with coated steel superstructure elements	Defect 3440 of painted steel Superstructure elements (various)	90% of steel superstructure elements in condition state 2 or better (effective)
Maintain bearings	95 % of bridges with bearings	Bearing elements (various)	90% of bearings in condition state 2 or better
Seal concrete decks	20% of eligible concrete decks each year	Element 8000 and 8514	Concrete sealer applied

Table 42.4-1
Objectives and Performance Measures

42.4.4 Preservation Program Benefits

Each objective and measure proposed in Table 42.4-1 is aimed at extending the life of the main bridge components by performing timely cyclical or condition-based (corrective) preservation actions. The cost of performing preservation actions is minor when compared to premature replacement or rehabilitation of bridge components. The benefits of each objective are discussed below:

- Maintaining bridge decks in good or fair condition is an asset management approach that should extend the service life of bridges and promote the MAP21 objectives. Experience has shown that bridges designed for a 100-year life expectancy should have decks that last 55 years with progressive preservation activities though the life of the bridge deck. Appropriate corrective actions taken as part of deck preservation extends the bridge deck life significantly. The costs of such corrective actions are substantially less than the costs of prematurely replacing the decks.

- The objective of maintaining strip seal joints in good or fair condition will minimize the deterioration of bridge superstructure and substructure components, including girders, bearings, abutments and piers. There is significant cost each year in repairing structural elements that have deteriorated prematurely as a result of leaking joints. Maintaining effective (non-leaking) strip seal joints can significantly delay superstructure and substructure repair and replacement.

- Maintaining protective paint systems is important to avoid significant corrosion, especially at girder ends and in areas of salt spray from traffic. The structural components of the steel bridges may lose load carrying capacity if left unprotected. Protective paint coatings systems should have a service life of 25-40 years for the protection of structural steel. The objective of maintaining coated steel surfaces in good or fair condition will aim at creating a paint program for extending the life of steel components up to 100 years.

- Bridge bearings are a key component. Bearings support bridge superstructures and allow for expansion of the superstructure. Experience has shown that loss of lubrication, tipping, or corrosion of bearings can cause harm to the deck and superstructure. The proposed measure of keeping bearings in good or fair condition will help WisDOT maintain bridges in a state of good repair.

- Objective of sealing all eligible concrete decks at 5-year intervals will help delay deck deterioration and prolong deck life. Sealing decks every 3 to 5 years at a minor cost can delay deck deterioration by 10-12 years that will promote increased deck life.

42.5 Bridge Preservation Activities, Eligibility and Need Assessment Criteria

The bridge preservation activities shown below relate to deck, superstructure and substructure elements. Table 42.5-1 shows the most common bridge preservation activities that are considered cost effective when applied to the appropriate bridge at the appropriate time, as well as considered eligible for bridge preservation funding. Additionally, these activities together with the eligibility and prioritization criteria discussed in this section will form a basis to generate an eligibility list of bridges that are candidates for cyclical and condition based Preventative Maintenance actions.

Bridge Component	Bridge Preservation Type	Activity Description	Preventive Maintenance Type	Action Frequency (years)
All	Preventive Maintenance	Sweeping, power washing, cleaning	Cyclical	1-2
Deck	Preventive Maintenance	Deck washing	Cyclical	1
		Deck sweeping		1
		Deck sealing/crack sealing		3-5
		Thin polymer (epoxy) overlays		7-15
		Drainage cleaning/repair	Condition Based	As needed
		Joint cleaning		
		Deck patching		1- 2
		Chloride extraction		1 -2
		Asphalt overlay with membrane		5-15
		Polymer modified asphalt overlay		10-15
		Joint seal replacement		10
		Drainage cleaning/repair		1
	Repair or Rehab Element	Rigid concrete overlays	Condition Based	As needed
		Structural reinforced concrete overlay		
		Deck joint replacement		
		Eliminate joints		
Super	Preventive Maintenance	Bridge approach restoration	Cyclical	2
		Seat and beam ends washing		2
	Repair or Rehab Element	Bridge rail restoration	Condition Based	As needed
		Retrofit rail		
		Painting		
		Bearing restoration (replacement, cleaning, resetting)		
		Superstructure restoration		
		Pin and hanger replacement		
		Retrofit fracture critical members		
Sub	Preventive Maintenance	Substructure restoration	Condition Based	As needed
		Scour counter measure		
		Channel restoration		

Table 42.5-1
Bridge Preservation Activities

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

NBI Item	Element	NBI Criteria	Defect	Element Defect Condition State Criteria	Repair Action	Potential Benefits to NBI or CS	Anticipated Service Life Years
Deck	Joints	Item 58 ≥ 5	2350	CS2, CS3, or CS4	Joint Cleaning	CS1or CS2	
			2310	CS3 + CS4 ≥ 10%	Joint Seal Replacement/Restoration	CS1	5 to 8
			2360	CS3 + CS4 ≥ 25%	Joint Replacement ④ ⑦	CS1	10 to 20
				All Condition State	Joint Elimination ④	Elimination	15 to 25
	Railing	Item 58 ≥ 5		CS3 or CS4	Railing Restoration	CS1 or CS2	3 to 10
				CS3 or CS4	Railing Replacement/Retrofit ⑧	CS1	10 to 20
Super	Steel Elements	Item 59 ≥ 5		N/A	Superstructure Washing/Cleaning	NA	1 to 2
			3440	CS2 + CS3 Area> 5% ⑥	Painting - Spot	CS1	1 to 5
				CS3 Area ≤ 25% ⑥	Painting - Zone	CS1 ①	5 to 7
				CS3 Area ≥ 25% ⑥	Painting - Complete	CS1 ②	15 to 20
		Item 59 ≥ 4		CS2, CS3, or CS4	Superstructure Restoration ③	NBI ≥ 7	5 to 20
	Bearings	Item 59 ≥ 5		CS3 or CS4	Bearing Reset/Repair	CS1 or CS2	1 to 5
				CS2 or CS3	Bearing Cleaning/Painting	CS1 or CS2	5 to 7
				CS3 + CS4 ≥ 25% or CS4 > 5%	Bearing Replacement	CS1	10 to 15
Sub	Miscellaneous	Item 60 ≥ 5		N/A	Substructure Washing/Cleaning	NA	1 to 2
			3440	CS2+CS3+CS4 Area > 5% ⑥	Painting - Spot	CS1	1 to 5
			3440	CS3 Area > 25% ⑥	Painting - Complete	CS1 ②	10 to 20
				CS2 or CS3 or CS4	Substructure Restoration ⑤	NBI ≥ 7	5 to 20
			9290	CS1 or CS2	Pier Protection ⑨	NBI ≥ 7	5 to 20
				CS3 or CS4	Scour Counter Measure ⑩	NBI ≥ 7	5 to 20

Table 42.5-3
Other Bridge Elements Eligibility Matrix

① Increase NBI only if combine with structural steel repairs.

② Complete painting only if combined with structural steel repairs to improve the component NBI ≥ 7.

③ Superstructure restoration includes all work related to the superstructure including but not limited to strengthening, pin and hanger replacement, retrofit FC member, etc.

④ Combined with deck overlay or replacement project.

⑤ Substructure restoration includes all work related to the substructure including but not limited to fiber wrapping, strengthening, crack injection, encapsulation, etc.—regardless of material type.

⑥ Element condition state for steel protective coating.

⑦ Includes but is not limited to end block/paving block replacement.

⑧ Must bring railing to current standards or have an approved exception to standards.

⑨ Examples are pier protection dolphins and fender systems.

⑩ Provide scour countermeasures after repairing any other substructure defects.

42.5.2 Identification of Preservation Needs

The identification of preservation needs will start with inventory and inspection information collected as part of the ongoing inspection program. The inspection information is analyzed by BOS asset management engineers with the WiSAMS (Chapter 41.2.1). The analysis includes current and past inspection reports, projected condition of each bridge component and element, past work actions, and preservation policy logic as shown in Table 42.5-2 and Table 42.5-3.

The programming of projects will start with the development of eligible structures work concepts as defined in Chapter 41 – Structures Asset Management. Eligible work could be standalone projects or combined into roadway projects, or combined into a group that may include cyclical preventive maintenance activities. Programming of work will be through the Improvement (Let) program and various Maintenance programs (DMA, RMA, and PBM)

Code	Description	Common Actions
9	EXCELLENT CONDITION - Isolated inherent defects.	Preservation/Cyclic Maintenance
8	VERY GOOD CONDITION - Some inherent defects.	
7	GOOD CONDITION - Some minor defects.	
6	SATISFACTORY CONDITION - Widespread minor or isolated moderate defects.	Preservation/ Condition-Based Maintenance
5	FAIR CONDITION- Some moderate defects; strength and performance of the component are not affected.	
4	POOR CONDITION - Widespread moderate or isolated major defects; strength and/or performance of the component is affected.	Rehabilitation or Replacement
3	SERIOUS CONDITION - Major defects; strength and/or performance of the component is seriously affected. Condition typically necessitates more frequent monitoring, load restrictions, and/or corrective actions.	
2	CRITICAL CONDITION - Major defects; component is severely compromised. Condition typically necessitates frequent monitoring, significant load restrictions, and/or corrective actions in order to keep the bridge open.	
1	IMMINENT FAILURE CONDITION - Bridge is closed to traffic due to component condition. Repair or rehabilitation may return the bridge to service.	
0	FAILED CONDITION - Bridge is closed to traffic due to component condition, and is beyond corrective action. Replacement is required to restore service.	

Table 42.7-1
NBI General Condition Ratings & Common Actions[4]

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

42.8 References

1. *Bridge Preservation Guide*, *Maintaining a Resilient Infrastructure to Preserve Mobility* (FHWA) – Spring 2018, (https://www.fhwa.dot.gov/bridge/preservation/guide/guide.pdf)

2. FDM 3-1 Exhibit 5.2 Agreement for the Use of Federal Funds for Preventive Maintenance of Structures. (May 2016). (https://wisconsindot.gov/rdwy/fdm/fd-03-05-e0502.pdf#fd3-5e5.2)

3. Source: U.S. DOT Secretary Mary Peters July 25, 2008 letter to Congress

4. Specifications for the National Bridge Inventory. FHWA-HIF-22-017 (March 2022) www.fhwa.dot.gov/bridge/snbi/snbi_march_2022_publication.pdf

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Loading Type	Live Load Factor
AASHTO Legal Vehicles, State Specific Vehicles, and Lane Type Legal Load Models	1.45
Specialized Haul Vehicles (SU4, SU5, SU6, SU7)	1.45
FAST Act Emergency Vehicles (EV2, EV3) **Alternate load factors per NCHRP Project 20-07/Task 410 are allowed.*	1.30*

Table 45.3-2
Live Load Factors (γ_{LL}) for Legal Loads in LRFR

Permit Type	Loading Condition	Distribution Factor	Live Load Factor
Annual	Mixed with Normal Traffic	Two or more lanes	1.30
Single Trip	Mixed with Normal Traffic	One Lane	1.20
Single Trip	Escorted with no other vehicles on the bridge	One Lane	1.10

Table 45.3-3
Live Load Factors (γ_{LL}) for Permit Loads in LRFR

45.3.7.3 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 for concrete and steel structures are presented in Section 17.2.6, and resistance factors for timber structures are presented in **MBE [6A.7.3]**.

45.3.7.4 Condition Factor: ϕ_C

The condition factor provides a reduction to account for the increased uncertainty in the resistance of deteriorated members and the likely increased future deterioration of these members during the period between inspection cycles.

WisDOT policy items:

Current WisDOT policy is to set the condition factor equal to 1.0.

45.3.7.5 System Factor: ϕ_S

System factors are multipliers applied to the nominal resistance to reflect the level of redundancy of the complete superstructure system. Bridges that are less redundant will have their factor member capacities reduced, and, accordingly, will have lower ratings. The aim of the system factor is to provide reserve capacity for safety of the traveling public. See Table 45.3-4 for WisDOT system factors.

Superstructure Type	ϕ_S
Welded Members in Two-Girder/Truss/Arch Bridges	0.85
Riveted Members in Two-Girder/Truss/Arch Bridges	0.90
Multiple Eyebar Members in Truss Bridges	0.90
Three-Girder Bridges with Girder Spacing ≤ 6.0 ft	0.85
Four-Girder Bridges with Girder Spacing ≤ 4.0 ft	0.95
All Other Girder and Slab Bridges	1.00
Floorbeam Spacings > 12.0 ft and Non-Continuous Stringers	0.85
Redundant Stringer Subsystems Between Floorbeams	1.00

Table 45.3-4
System Factors for WisDOT

45.3.7.6 Design Load Rating

The design load rating assesses the performance of bridges utilizing the LRFD design loading, producing an inventory and operating rating. Note that when designing a new structure, it is required that the RF > 1.0 at the inventory level. In addition to providing a relative measure of bridge capacity, the design load rating also serves as a screening process to identify bridges that should be load rated for legal loads. If a structure has an inventory RF < 0.9, it may not be able to safely carry emergency vehicles, and if it has an operating RF < 1.0, it may not be able to safely carry other legal-weight traffic and therefore a legal load rating must be performed. If a structure has rating factors above these thresholds, proceeding to the legal load rating is not required. However, the load rating engineer is still required to rate the Wisconsin Standard Permit Vehicle (Wis-SPV) as shown in 45.12.

45.3.7.6.1 Design Load Rating Live Load

The LRFD design live load, HL-93, shall be utilized as the rating vehicle(s). The components of the HL-93 loading are described in 17.2.4.2.

45.3.7.7 Legal Load Rating

Bridges that do not satisfy the HL-93 design load rating check (RF < 1.0 at operating level) shall be evaluated for legal loads to determine if legal-weight traffic should be restricted; whether a load posting is required. Additionally, bridges that do not satisfy the HL-93 design load rating check (RF < 0.9 at inventory level) shall be evaluated for FAST Act emergency vehicle loads to determine if emergency vehicle-specific weight limits are required. If the load

45.8 WisDOT Load Rating Policy and Procedure – Culverts

45.8.1 Culvert Rating Methods

Bridge-length culverts (assigned a B- or P-number) shall be load rated according to one of the following methods:

- Calculated (LFR or LRFR)
- Assigned
- Field Evaluation and Documented Engineering Judgment

Calculated ratings are preferred. However they have not been required historically, and many culverts are designed based on minimum standards, while being relatively low-risk for failure. Therefore, assigned ratings or field evaluation and documented engineering judgment are acceptable methods for culverts meeting criteria described in the following sections.

For non-bridge-length culverts (assigned a C-number):

- New culverts shall be load rated the same as bridge-length culverts.
- For existing (in-service) culverts being rehabilitated, a load rating update is required only if a loading change would reduce the culvert's live load capacity below its original design load level. When load rating is not required, report ratings taken from HSI and the date. Contact the Bureau of Structures Rating Unit to discuss load rating existing (in-service) culverts prior to plan submittal.
- For culvert extensions, the new extended portion shall follow the above requirements for new culverts, and the existing portion shall follow the above requirements for rehabilitation of culverts. When different load rating methods are used for the new and existing portions of an extended culvert, provide ratings for both, as described in 6.2.2.3.4.
- For existing (in-service) culverts not being rehabilitated, a load rating update is not required. However, if deterioration or other significant changes warrant consideration of a load posting, contact the Bureau of Structures Rating Unit for evaluation requirements.

45.8.2 Rating New Culverts

Concrete box culverts shall have load ratings calculated per AASHTO specifications, using LRFR methodology with HL-93 loading and inclusive of the Wisconsin Standard Permit Vehicle (Wis-SPV).

Other culvert types are more commonly designed based on manufacturers' tables for size, fill depth, and design load. Therefore, load ratings may be either calculated or assigned. If load ratings are calculated, they shall be reported on plans. Assigned load ratings must have stamped plans and/or design calculations indicating design load and fill depth. As a minimum, they shall be designed to carry HL-93 or HS20 loading and the Wis-SPV as described in 36.1.3. Assigned load ratings shall be reported as:

Design Vehicle	Inventory	Operating	Wis-SPV
HS20	HS20	HS33	190 k
HL93	RF1.00	RF1.30	190 k

Table 45.8-1
Assigned Load Ratings for New Culverts Other than Concrete Boxes

45.8.3 Rating Existing (In-Service) Culverts

The load rating method for existing (in-service) bridge-length culverts shall be determined based on culvert type, design load and method, fill depth, condition, and availability of known construction details. Refer to the following sections for more guidance and see 45.9 for documentation and submittal requirements.

45.8.3.1 Assigned Ratings for In-Service Culverts

The Bureau of Structures allows the use of assigned load ratings for culverts based on the FHWA Memo dated September 29th, 2011. Furthermore, the Bureau of Structures has conducted parametric studies to extend the application of assigned load ratings to additional older design loads and methods and to include additional vehicles. Assigned load ratings may be used if all of the following are true:

- Engineer-stamped or -signed plans or design calculations are on file, with the original design load and fill depth clearly indicated,
- Current fill depth is within 12 inches of original design fill depth range, and no other load changes have occurred that could reduce the inventory rating below the original design load level,
- Structural members have no appreciable signs of distress or deterioration that would affect structural capacity, and
- Culvert type, design load, and design method are among the combinations listed in Table 45.8-2 that allow assigned load ratings. This table was developed by Bureau of Structures based on WisDOT culverts.

Culvert Type	Design Load	Design Method	Inventory	Operating	EV2 RF	EV3 RF	Wis-SPV
All	HL93	LRFD	RF1.00	RF1.30	N/A	N/A	190 k
All	HS20	LFD	HS20	HS33	N/A	N/A	190 k
Concrete Box	H20[(a)], HS20	ASD	HS16	HS27	1.20	1.00	170 k

(a) If designed for H20 per 1957 (or earlier) AASHTO design specification and designed for fill depth less than 2.0', load ratings shall be calculated (assigned ratings cannot be used).

Table 45.8-2
Assigned Load Ratings for In-Service Bridge-Length Culverts

45.8.3.2 Calculated Ratings for In-Service Culverts

Calculated load ratings are preferred when as-built plans or field measurements with necessary load rating parameters are available. They are required if sufficient construction details are known and the culvert does not qualify for assigned load ratings per 45.8.3.1.

An exception is allowed when the fill depth is 10'-0" or greater. At this depth, live load effects are negligible, and field evaluation and documented engineering judgment per 45.8.3.3 may be used.

Top slab flexure is expected to be the controlling limit state for calculated load ratings. However, some older culverts may have low calculated ratings due to conservative methods for shear, bottom slab flexure, or other limit states and locations. Upon consultation with Bureau of Structures, consideration may be given to ignoring these rating checks when the final load ratings are reported, if the culvert does not show signs of distress.

45.8.3.3 Engineering Judgment Ratings for In-Service Culverts

When assigned or calculated load ratings cannot be used (typically due to unknown construction details or severe deterioration effects that cannot be quantified), or when the depth of fill is 10'-0" or greater, the load rating may be determined via field evaluation and documented engineering judgment. Table 45.8-3 may be used as a general guide. This table was developed by Bureau of Structures based on WisDOT culverts. Contact Bureau of Structures immediately for any culvert condition in which a weight limit posting may be warranted.

<table>
<tr><th>NBI Culvert Condition Rating</th><th>Fill Depth</th><th>Element in CS4 Under Traffic Lanes?</th><th>Inventory</th><th>Operating</th><th>Wis-SPV</th><th>Weight Limit Restriction</th></tr>
<tr><td>≥ 5</td><td>N/A</td><td>N/A</td><td>HS20[a]</td><td>HS33</td><td>190 k</td><td>NONE</td></tr>
<tr><td>4</td><td>N/A</td><td>N/A</td><td>HS12</td><td>HS20</td><td>170 k</td><td>NONE</td></tr>
<tr><td rowspan="3">3</td><td>≥ 10’</td><td>N/A</td><td>HS12</td><td>HS20</td><td>170 k</td><td>NONE</td></tr>
<tr><td rowspan="2">< 10’</td><td>No</td><td>HS12</td><td>HS20</td><td>170 k</td><td>NONE</td></tr>
<tr><td>Yes</td><td>HS06</td><td>HS10</td><td>40 k</td><td>20 TON</td></tr>
<tr><td rowspan="3">2</td><td>≥ 10’</td><td>N/A</td><td>HS12</td><td>HS20</td><td>170 k</td><td>NONE</td></tr>
<tr><td rowspan="2">< 10’</td><td>No</td><td>HS06</td><td>HS10</td><td>40 k</td><td>20 TON</td></tr>
<tr><td>Yes</td><td>HS02</td><td>HS03</td><td>10 k</td><td>5 TON</td></tr>
<tr><td>0-1</td><td>N/A</td><td>N/A</td><td>HS00</td><td>HS00</td><td>0</td><td>CLOSE</td></tr>
</table>

(a) If design load less than HS20 is known or reasonably assumed, the inventory rating may be set equal to the design load. H15 design shall be considered equal to HS15 and H20 design may be considered equal to HS20. Operating Rating should be estimated as 1.67 x Inventory Rating.

Table 45.8-3

Engineering Judgment Load Ratings for In-Service Culverts

If rating factors need to be recorded for posting or emergency vehicles for National Bridge Inventory data, they shall be calculated as (Weight Limit Restriction) / (Vehicle Weight) if a weight limit restriction exists, otherwise 1.0. The Load Rating Summary Sheet shall include a note indicating assumed rating factor values were recorded.