## DISCLAIMER

Although the material in this Wisconsin Bridge Manual has been tested by the Bureau of Structures, no warranty, expressed or implied, is made by the Wisconsin Department of Transportation, as to the accuracy of the material in this manual, nor shall the fact of distribution constitute any such warranty, and responsibility is not assumed by Wisconsin Department of Transportation in connection therewith.

## 1.1 Introduction

The Bridge Manual is for the guidance of design engineers, technicians, and inspection personnel engaged in bridge design, plan preparation, and construction for the Wisconsin Department of Transportation. It is prepared to encourage uniform application of designs and standard details in plan preparation of bridges and other related structures.

This manual is a guide for the layout, design and preparation of highway structure plans. It does not replace, modify, or supersede any provisions of the Wisconsin Standard Specifications, plans or contracts.

## 1.2 Index

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## 3.1 Specifications and Standards

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

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

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

## 3.2 Geometrics and Loading

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

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

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

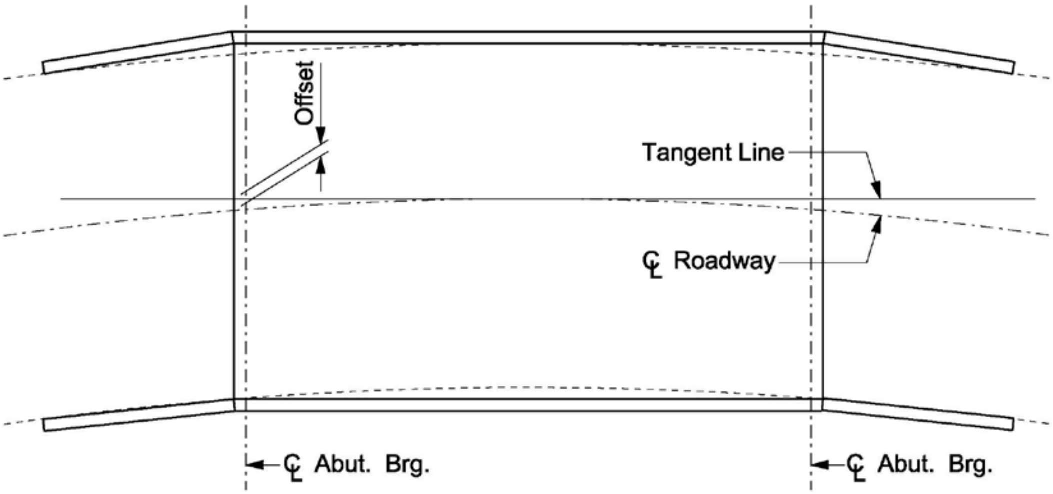
Highway bridge design live loadings follow the AASHTO LRFD Design specifications using HL93. Chapter 17 provides more detail on applying this load for design. WisDOT requires a specific vehicle design check using the Wis-SPV (Standard Permit Vehicle) which can be found in Chapter 45.

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

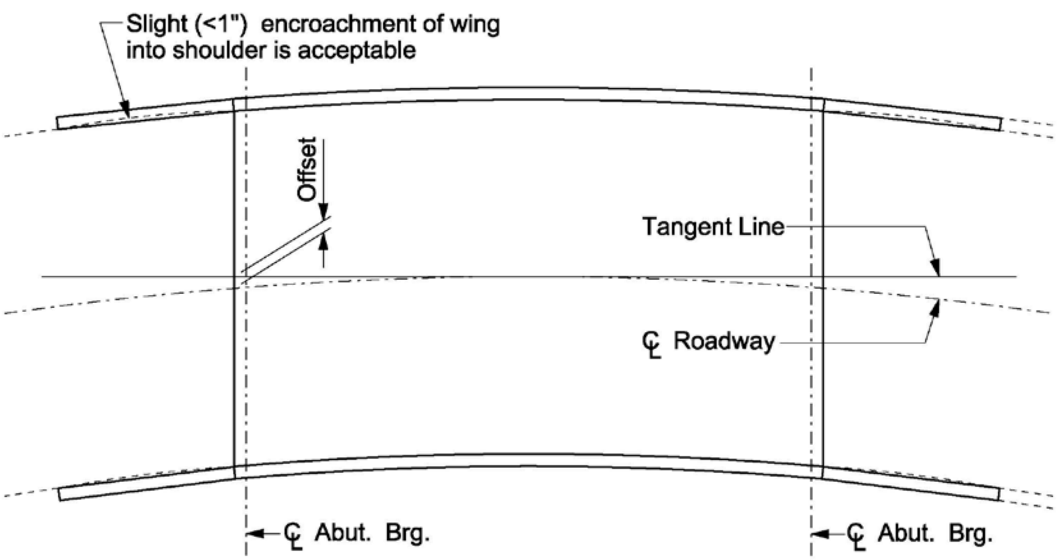
All new bridges constructed in the State of Wisconsin are designed for the clearances shown in FDM Procedure 11-35-1, Attachment 8. FDM Procedure 11-35-1, Attachment 9 covers the cases described in that section as well as bridge widenings. Wires and cables over highways are designed for clearances of 18’-0” to 22’-0”. Vertical clearance is needed for the entire roadway width (critical point to include traveled way, auxiliary lanes, turn lanes and shoulders).

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

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




**Case 1**
For offsets 0" to 6"
Keep bridge straight. Widen bridge to provide full lane and shoulder width over entire length of bridge (round up to nearest 1"). Align straight wings so inside of wing tip is at edge of shoulder.




**Case 2**
For offsets over 6"
Curve entire bridge. Do not widen. Align straight wings so inside of wing tip is at edge of shoulder.

**Figure 3.2-1**
Bridge Layout on Horizontal Curves

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## 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 much 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.

## 4.4 Secondary Features

Color

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

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

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

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

Pattern and Texture

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

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

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

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

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

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

Ornamentation

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

Regarding ornamentation in general, more is seldom better.

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

- J.B. Johnson, 1912

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7. Pile Plan & Splice Detail

8. View Showing Limits of Excavation and Backfill

9. Special Details for Utilities

10. Drainage Details

### 6.3.2.4 Piers

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

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

1. Plan View

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

2. Elevation

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

3. Footing Plan

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

4. Bar Steel Listing and Details

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

6. Cross Section thru Column and Pier Cap

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

### 6.3.2.5 Superstructure

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

6.3.2.5.1 All Structures

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

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

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

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

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

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

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

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

**Table 6.3-2**
Various Inspection Reports

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

## 6.3.5 Processing Plans

1. Before P.S. & E. Process

   File plans in plan drawers by county for consultant work, or

   Maintain plans as PDF on E-plan server.

2. At P.S. & E. Processing

   Prepare plans for bid letting process.

3. After Structure Construction

   Any data in Design Folder is scanned and placed with bridge plans.

   Original plan sheets and Design Folders are discarded.

## 6.4 Computation of Quantities

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

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

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

Staged Construction - On projects where there is staged construction that will involve two construction seasons the following quantities should be split to match the staging to aid the contractor/fabricator: Concrete Masonry, Bar Steel Reinforcement, Structural Steel and Bar Couplers. The other items are not significant enough to justify separating.

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

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

### 6.4.1 Excavation for Structures Bridges (Structure)

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

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

### 6.4.2 Backfill Granular or Backfill Structure

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

### 6.4.3 Concrete Masonry Bridges

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

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## 9.1 General

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

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

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

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

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

retaining walls, hooks may be the only practical solution because of the concrete depth available for developing the capacity of the bars.

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

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

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

## 9.3.1 Development Length and Lap Splices for Deformed Bars

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

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

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

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

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

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

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

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

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

## 9.3.2 Bends and Hooks for Deformed Bars

Figure 9.9-1 shows standard hook and bend details for development of longitudinal tension reinforcement. Figure 9.9-2 shows standard hook and bend details for transverse reinforcement (open stirrups and ties). Figure 9.9-3 shows details for transverse reinforcement (closed stirrups). Dimensions for the bending details are shown as out to out of bar, as stated in the *Standard Specifications* Section 505.3.2. The diameter of a bend, measured on the inside of the bar for a standard bend is specified in **LRFD [5.10.2.3]**. Where a larger bend radius is required (non-standard bend) show the inside bend radius on the bar detail. When computing total bar lengths account for the accumulation in length in the bends. Use the figures mentioned above to account for accumulation in length for standard hooks and bends. One leg of bent bars is not dimensioned so that the tolerance for an error in the length due to bending is placed there. Fabrication tolerances for bent bars are specified in the *Concrete Reinforcing Steel Institute (CRSI) Manual of Standard Practices* or the *American Concrete Institute (ACI) Detailing Manual* as stated in Section 505.2.1 of the *Standard Specifications*.

## 9.8 Painting

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

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

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

**Table 9.8-1**
Standard Colors for Steel Girders

[1] Shop applied color for weathering steel.

Federal Standard No. 595B can be found at www.colorserver.net/

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

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

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

The primary reason for painting steel structures is for the protection of the steel surface. Appearance is a secondary function that is maintained by using compatible top coatings over epoxy systems.

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

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

Refer to Section 1.3.14 of the *Wisconsin Structure Inspection Manual* for the criteria covering spot painting versus complete painting of existing structures. This Section provides information for evaluating the condition of a paint system and recommended maintenance.

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

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

**Table 9.8-2**
Standard Colors for Concrete

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

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Footnotes to Figure 12.7-1:

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

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

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

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

## 12.8 Abutment Design Loads and Other Parameters

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

### 12.8.1 Application of Abutment Design Loads

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

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

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

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

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

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

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

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

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

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

**WisDOT policy item:**

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

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

### 12.8.4 Other Abutment Design Parameters

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

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

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

**Table 12.8-4**
Other Parameters Used in Abutment Design

### 12.8.5 Abutment and Wing Wall Design in Wisconsin

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

**WisDOT policy items:**

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

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

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

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

## 12.8.6 Horizontal Pile Resistance

The following procedure shall be used to verify the horizontal resistance of the piles for A3 and A4 abutments.

Given information:

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

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

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

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

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

$Hr_{battered}$ = 38.8 kips per pile

Calculate ultimate resistance provided by the pile configuration:

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

Hr = 10.4 klf

Hr > Hu = 10.3 klf OK

## 12.9 Abutment Body Details

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

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

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

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

### 12.9.1 Construction Joints

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

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

**WisDOT exception to AASHTO:**

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

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

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

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

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

- Use of select materials

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

- Strict control of moisture and density

- Proper compaction

- Installation of moisture barriers

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

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

The bridge designer should determine if a structural approach slab is required and coordinate details with the roadway engineer. Usage of structural approach slabs is currently based on road functional classifications and considerations to traffic volumes (AADT), design speeds, and settlement susceptibility.

**WisDOT policy item:**

Structural approach slabs shall be used on all Interstate Highway bridges and U.S.H. bridges. Other locations can be considered with the approval of the Chief Structural Design Engineer.

Standards for Structural Approach Slab for Type A1, A3, and A4 Abutments and Structural Approach Slab Details for Type A1, A3, and A4 Abutments are available for guidance.

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- Non-proprietary walls (e.g., cast-in-place, sheet pile, and all other wall types other than those previously referenced):
    - Walls having an exposed height of less than 5.5 ft. measured from the plan ground line to top of wall may require an R number based on specific project features. Designer to contact the Bureau of Structures region liaison for more information.

Cast-in-place walls being utilized strictly as bridge abutment or box culvert wings do not require R numbers as they are considered part of the structure.

## 14.2 Wall Types

Retaining walls can be divided into many categories as discussed below.

Conventional Walls

Retaining walls can be divided into gravity, semi-gravity, and non-gravity cantilever or anchored walls. A brief description of these walls is presented in 14.2.1 and 14.2.2 respectively.

Miscellaneous types of walls including multi-tiered walls, and hybrid or composite walls are also used by combining the wall types mentioned in the previous paragraph. These walls are used only under special project requirements. These walls are briefly discussed in 14.2.3, but the design requirements of these walls will not be presented in this chapter. In addition, some walls are also used for temporary shoring and discussed briefly in 14.2.4.

Permanent or Temporary Walls

All walls can be divided into permanent or temporary walls, depending on project application. Permanent walls have a typical designed life of 75 years. The temporary walls are designed for a service life of 3 years, or the intended project duration, whichever is greater. Temporary wall systems have less restrictive requirements for construction, material and aesthetics.

Fill Walls or Cut Walls

A retaining wall can also be classified as a fill wall, or a cut wall. This description is based on the nature of the earthwork required to construct the wall. If the roadway cross-sections (which include the wall) indicate that existing earth/soil must be removed (excavated) to install the wall, it is considered a ‘cut’ wall. If the roadway cross-sections indicate that earth fill will be placed behind the wall, with little excavation, the wall is considered a ‘fill’ wall. Sometimes wall construction requires nearly equal combinations of earth excavation and earth fill, leading to the nomenclature of a ‘cut/fill’ wall.

Bottom-up or Top-down Constructed Walls

This wall classification method refers to the method in which a wall is constructed. If a wall is constructed from the bottom of the wall, upward to the top, it is considered a bottom-up type of wall. Examples of this include CIP cantilever, MSE and modular block walls. Bottom-up walls are generally the most cost effective type. If a wall is constructed downward, from the top of the wall to the bottom, it is considered a top-down type of wall. This generally requires the insertion of some type of wall support member below the existing ground, and then excavation in front of the wall to the bottom of the exposed face. Examples of this include soil nail, cantilever sheet pile and anchored sheet pile walls. These walls are generally used when excavation room is limited.

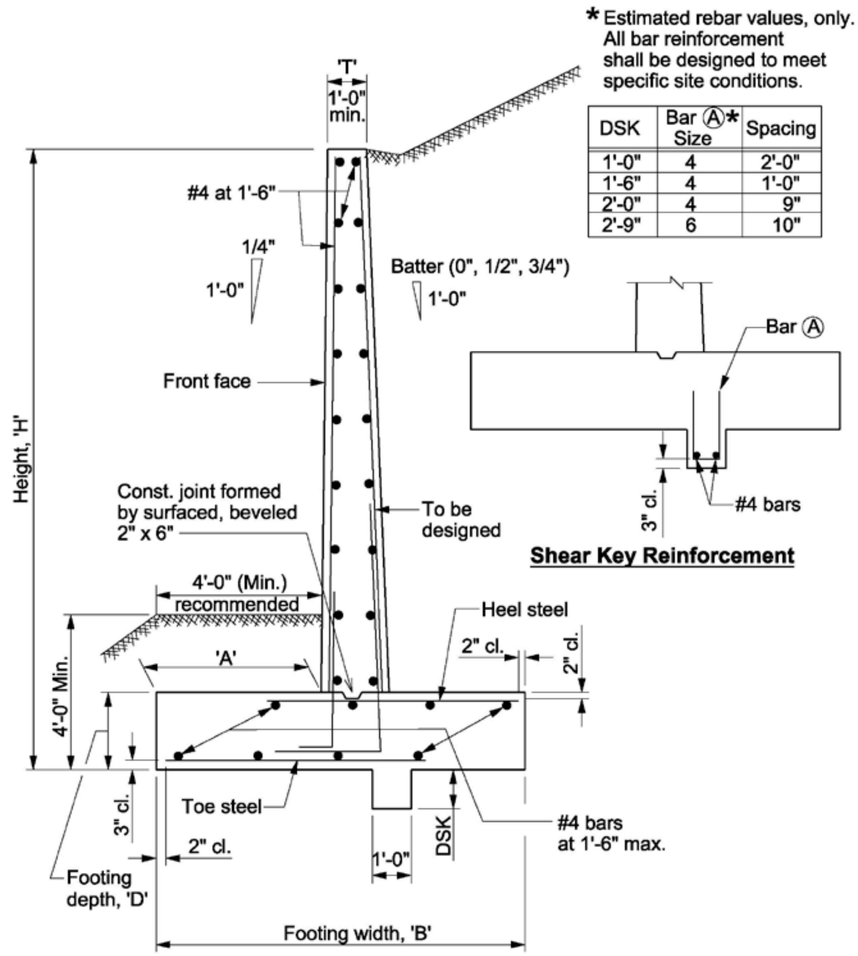


**Figure 14.5-2**
CIP Walls General Details

## 14.5.3.1 Wall Back and Front Slopes

CIP walls shall not be designed for backfill slope steeper than 2:1(H:V). Where practical, walls shall have a horizontal bench of 4.0 feet wide at the front face.

## 14.5.4 Unfactored and Factored Loads

Unfactored loads and moments are computed after establishing the initial wall geometry and using procedures defined in 14.4.5.4.5. A load diagram as shown in Figure14.4-1 for the earth pressure is developed assuming a triangular distribution plus additional pressures resulting from earth surcharge, water pressure, compaction or any other loads, etc. The material properties for backfill soil, concrete and steel are given in 14.4.6. The foundation and retained earth properties as recommended in the Geotechnical Report shall be used for computing nominal loads.

The computed nominal loads discussed in 14.5.4 are multiplied by applicable load factors given in Table 14.4-1. Figure 14.4-8 shows load factor and load combinations along with their application for the load limit state evaluation. A summary of load factors and load combinations as applicable for a typical CIP cantilever wall is presented in Table 14.4-1 and **LRFD [3.4.1]**, respectively. Computed factored loads and moments are used for performing stability checks.

## 14.5.5 External Stability Checks

The external stability check includes checks for limiting eccentricity (overturning), bearing stress, and sliding at Strength I and Extreme Event II due to vehicle impact in cases where live load traffic is carried.

### 14.5.5.1 Eccentricity Check

The retaining wall shall be evaluated for the eccentricity. The location of the resultant force should be within the middle half of the base width (e<B/4) of the foundation centroid for foundations on soil, and within the middle three-fourths of the base width (e< 3B/8) of the foundation centroid for foundations on rock. If there is inadequate resistance to overturning (eccentricity value greater than limits given above), consideration should be given to either increasing the width of the wall base, or providing a deep foundation.

### 14.5.5.2 Bearing Resistance

The Bearing resistance shall be evaluated at the strength limit state using factored loads and resistances. Bearing resistance of the walls founded directly on soil or rock shall be computed in accordance with 11.2 and **LRFD [10.6]**. The Bearing Resistance for walls on piles shall be computed in accordance with 11.3 and **LRFD [10.6]**. Figure 14.5-3 shows bearing stress criteria for a typical CIP wall on soil and rock respectively.

The vertical stress for footings on soil shall be calculated using:

$$\sigma_v = \frac{\sum V}{(B-2e)}$$

For walls founded on rock, the vertical stress is calculated assuming a linearly distributed pressure over an effective base area. The vertical stress for footings on rock shall be computed using:

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

## 14.14 Contract Plan Requirements

The following minimum information shall be required on the plans.

1. Finish grades at rear and front of wall at 25 foot intervals or less.

2. Final cross sections as required for wall designer.

3. Beginning and end stations of wall and offsets from reference line to front face of walls. If reference line is a horizontal curve give offsets from a tangent to the curve.

4. Location of right-of-way boundaries, and construction easements relative to the front face of the walls.

5. Location of utilities if any and indicate whether to remain in place or be relocated or abandoned.

6. Special requirements on top of wall such as copings, railings, or traffic barriers.

7. Footing or leveling pad elevations if different than standard.

8. General notes on standard insert sheets.

9. Soil design parameters for retained soil, backfill soil and foundation soil including angle of internal friction, cohesion, coefficient of sliding friction, groundwater information and ultimate and/or allowable bearing capacity for foundation soil. If piles are required, give skin friction values and end bearing values for displacement piles and/or the allowable steel stress and anticipated driving elevation for end bearing piles.

10. Soil borings.

11. Details of special architectural treatment required for each wall system.

12. Wall systems, system or sub-systems allowed on projects.

13. Abutment details if wall is component of an abutment.

14. Connection and/or joint details where wall joins another structure.

15. Groundwater elevations.

16. Drainage provisions at heel of wall foundations.

17. Drainage at top of wall to divert run-off water.

## 14.15 Construction Documents

### 14.15.1 Bid Items and Method of Measurement

Proprietary retaining walls shall include all required bid items necessary to build the wall system provided by the contractor. The unit of measurement shall be square feet and shall include the exposed wall area between the footing and the top of the wall measured to the top of any copings. For setback walls the area shall be based on the walls projection on a vertical plane. The bid item includes designing the walls preparing plans, furnishing and placing all materials, including all excavations, temporary bracing, piling, (including delivered and driven), poured in place or precast concrete or blocks, leveling pads, soil reinforcement systems, structural steel, reinforcing steel, backfills and infills, drainage systems and aggregate, geotextiles, architectural treatment including painting and/or staining, drilled shafts, wall toppings unless excluded by contract, wall plantings, joint fillers, and all labor, tools, equipment and incidentals necessary to complete the work.

The contractor will be paid for the plan quantity as shown on the plans. (The intent is a lump sum bid item but is bid as square feet of wall). The top of wall coping is any type of cap placed on the wall. It does not include any barriers. Measurement is to the bottom of the barrier when computing exposed wall area.

Non-proprietary retaining walls are bid based on the quantity of materials used to construct the wall such as concrete, reinforcing steel, piling, etc. These walls are:

- Cast-in-Place Concrete Cantilever Walls
- Post-and-Panel Walls
- Steel Sheet Piling Walls

### 14.15.2 Special Provisions

The Structures Design Section has Standard Special Provisions for:

- Wall Modular Block Gravity LRFD, Item SPV.0165.
- Wall Modular Block Mechanically Stabilized Earth LRFD, Item SPV.0165.
- Wall Concrete Panel Mechanically Stabilized Earth LRFD, Item SPV.0165.
- Wall Concrete Panel Mechanically Stabilized Earth LRFD/QMP, Item SPV.0165
- Wall CIP Facing Mechanically Stabilized Earth LRFD, Item SPV.0165.
- Wall Wire Faced Mechanically Stabilized Earth LRFD, Item SPV.0165.
- Wall Wire Faced Mechanically Stabilized Earth LRFD/QMP, Item SPV.0165.
- *Wall Gabion LRFD, SPV under development.*

- *Wall Modular Bin or Crib LRFD, SPV under development.*
- *Temporary Wall Wire Faced Mechanically Stabilized Earth LRFD, SPV under development.*

Note that the QMP Special Provisions should be used beginning with the December 2014 letting or prior to December 2014 letting at the Region's request.

The designer determines what wall systems(s) are applicable for the project. The approved names of suppliers are inserted for each eligible wall system. The list of approved proprietary wall suppliers is maintained by the Structures Development Section which is responsible for the Approval Process for earth retaining walls, 14.16.

## 14.16 Submittal Requirements for Pre-Approval Process

### 14.16.1 General

The following four wall systems require the supplier or manufacturer to submit to the Structural Design Section a package that addresses the items specified in paragraph C.

1. Modular Block Gravity Walls
2. MSE Walls with Modular Block Facings
3. MSE Walls with Precast Concrete Panel Facings
4. Modular Concrete Bin or Crib Walls

### 14.16.2 General Requirements

Approval of retaining wall systems allows for use of these systems on Wisconsin Department of Transportation (WisDOT) projects upon the manufacturer's certification that the system as furnished to the contractor (or purchasing agency) complies with the design procedures specified in the *Bridge Manual*. WisDOT projects include: State, County and Municipal Federal Aid and authorized County and Municipal State Aid projects in addition to materials purchased directly by the state.

The manufacturer shall perform all specification tests with qualified personnel and maintain an acceptable quality control program. The manufacturer shall maintain records of all its control testing performed in the production of retaining wall systems. These test records shall be available at all times for examination by the Construction Materials Engineer for Highways or designee. Approval of materials will be contingent upon satisfactory compliance with procedures and material conformance to requirements as verified by source and field samples. Sampling will be performed by personnel during the manufacture of project specific materials.

### 14.16.3 Qualifying Data Required For Approval

Applicants requesting Approval for a specific system shall provide three copies of the documentation showing that they comply with *AASHTO LRFD* and *WisDOT Standard Specifications* and the design criteria specified in the *Bridge Manual*.

1. An overview of the system, including system theory.
2. Laboratory and field data supporting the theory.
3. Detailed design procedures, including sample calculations for installations with no surcharge, level surcharge and sloping surcharge.
4. Details of wall elements, analysis of structural elements, capacity demand ratio, load and resistance factors, estimated life, corrosion design procedure for soil

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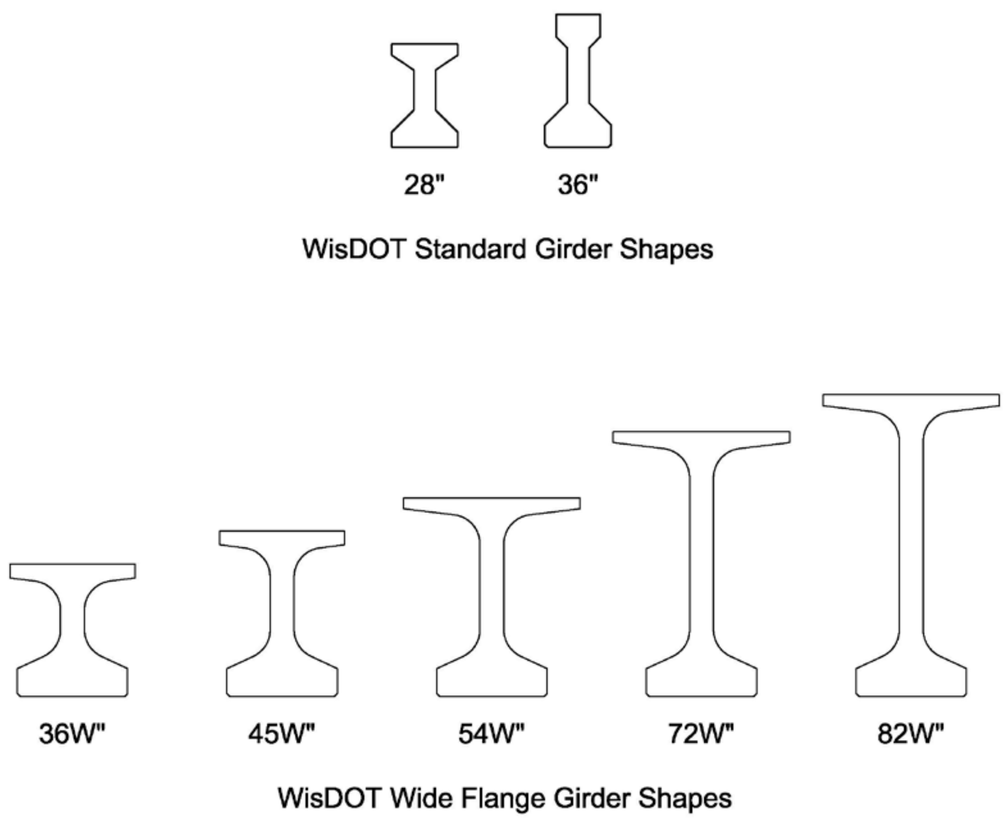
## 19.3.7 Construction Dimensional Tolerances

Refer to the *AASHTO LRFD Bridge Construction Specifications* for the required dimensional tolerances.

## 19.3.8 Prestressed Girder Sections

WisDOT BOS employs two prestress I girder section families. One I section family follows the AASHTO standard section, while the other I section family follows a wide flange bulb-tee, see Figure 19.3-7. These sections employ draped strand patterns with undraped alternates where feasible. Undraped strand patterns, when practical, should be specified on the designs. For these sections, the cost of draping far exceeds savings in strands. See the Standard Details for the I girder sections' draped and undraped strand patterns. Note, for the 28" prestressed I girder section the 16 and 18 strand patterns require bond breakers.




**<u>Figure 19.3-7</u>**
I Girder Family Details

Table 19.3-1 and Table 19.3-2 provide span lengths versus interior girder spacings for HL-93 live loading on single-span and multiple-span structures for prestressed I-girder sections. Girder spacings are based on using low relaxation strands at $0.75f_{pu}$, concrete haunch thicknesses, slab thicknesses from Chapter 17 – Superstructures - General and a future wearing surface. For these tables, a line load of 0.300 klf is applied to the girder to account for superimposed dead loads.

Several girder shapes have been retired from standard use on new structures. These include the following sizes; 45-inch, 54-inch and 70-inch. These girder shapes are used for girder replacements, widening and for curved new structures where the wide flange sections are not practical. See Chapter 40 – Bridge Rehabilitation for additional information on these girder shapes.

Due to the wide flanges on the 54W, 72W and 82W and the variability of residual camber, haunch heights frequently exceed 2”. An average haunch of 4” was used for all wide flange girders in the following tables. **Do not push the span limits/girder spacing during preliminary design.** See Table 19.3-2 for guidance regarding use of excessively long prestressed girders.

For interior prestressed concrete I-girders, 0.5” or 0.6” dia. strands (in accordance with the Standard Details).

$f'_c$ girder = 8,000 psi

$f'_c$ slab = 4,000 psi

Haunch height (dead load) = 2 ½” for 28” and 36” girders

= 4” for 45W”,54W”,72W” and 82W” girders

Haunch height (section properties) = 2”

Required $f'_c$ girder at initial prestress < 6,800 psi

| 28" Girder | | |
|---|---|---|
| Girder Spacing | Single Span | 2 Equal Spans |
| 6’-0” | 54 | 60 |
| 6’-6” | 54 | 58 |
| 7’-0” | 52 | 56 |
| 7’-6” | 50 | 54 |
| 8’-0” | 50 | 54 |
| 8’-6” | 48 | 52 |
| 9’-0” | 48 | 50 |
| 9’-6” | 46 | 50 |
| 10’-0” | 44 | 48 |
| 10’-6” | 44 | 48 |
| 11’-0” | 42 | 46 |
| 11’-6” | 42 | 46 |
| 12’-0” | 42 | 44 |

| 36" Girder | | |
|---|---|---|
| Girder Spacing | Single Span | 2 Equal Spans |
| 6’-0” | 72 | 78 |
| 6’-6” | 70 | 76 |
| 7’-0” | 70 | 74 |
| 7’-6” | 68 | 72 |
| 8’-0” | 66 | 70 |
| 8’-6” | 64 | 68 |
| 9’-0” | 62 | 68 |
| 9’-6” | 60 | 64 |
| 10’-0” | 60 | 64 |
| 10’-6” | 58 | 62 |
| 11’-0” | 50 | 60 |
| 11’-6” | 50 | 60 |
| 12’-0” | 48 | 58 |

| 36W" Girder | | |
|---|---|---|
| Girder Spacing | Single Span | 2 Equal Spans |
| 6’-0” | 94 | 101 |
| 6’-6” | 92 | 99 |
| 7’-0” | 88 | 97 |
| 7’-6” | 87 | 95 |
| 8’-0” | 85 | 93 |
| 8’-6” | 83 | 91 |
| 9’-0” | 82 | 88 |
| 9’-6” | 80 | 86 |
| 10’-0” | 77 | 84 |
| 10’-6” | 76 | 83 |
| 11’-0” | 74 | 81 |
| 11’-6” | 73 | 78 |
| 12’-0” | 71 | 76 |

| 45W" Girder | | |
|---|---|---|
| Girder Spacing | Single Span | 2 Equal Spans |
| 6’-0” | 110 | 120 |
| 6’-6” | 109 | 117 |
| 7’-0” | 107 | 115 |
| 7’-6” | 103 | 113 |
| 8’-0” | 101 | 111 |
| 8’-6” | 99 | 108 |
| 9’-0” | 97 | 104 |
| 9’-6” | 95 | 102 |
| 10’-0” | 92 | 100 |
| 10’-6” | 90 | 98 |
| 11’-0” | 88 | 96 |
| 11’-6” | 87 | 93 |
| 12’-0” | 85 | 91 |

**Table 19.3-1**
Maximum Span Length vs. Girder Spacing

| 54W" Girder | | |
|---|---|---|
| Girder Spacing | Single Span | 2 Equal Spans |
| 6'-0" | 125 | 134 |
| 6'-6" | 123 | 132 |
| 7'-0" | 120 | 129 |
| 7'-6" | 118 | 127 |
| 8'-0" | 116 | 125 |
| 8'-6" | 114 | 122 |
| 9'-0" | 112 | 120 |
| 9'-6" | 110 | 117 |
| 10'-0" | 108 | 115 |
| 10'-6" | 106 | 114 |
| 11'-0" | 102 | 111 |
| 11'-6" | 101 | 109 |
| 12'-0" | 99 | 107 |

| 72W" Girder | | |
|---|---|---|
| Girder Spacing | Single Span | 2 Equal Spans |
| 6'-0" | 153* | 164*⊗ |
| 6'-6" | 150 | 161*⊗ |
| 7'-0" | 148 | 158* |
| 7'-6" | 145 | 156* |
| 8'-0" | 143 | 153* |
| 8'-6" | 140 | 150 |
| 9'-0" | 138 | 148 |
| 9'-6" | 135 | 144 |
| 10'-0" | 133 | 142 |
| 10'-6" | 131 | 140 |
| 11'-0" | 129 | 137 |
| 11'-6" | 127 | 135 |
| 12'-0" | 124 | 132 |

| 82W" Girder | | |
|---|---|---|
| Girder Spacing | Single Span | 2 Equal Spans |
| 6'-0" | 166*⊗ | 177*⊗ |
| 6'-6" | 163*⊗ | 174*⊗ |
| 7'-0" | 161*⊗ | 172*⊗ |
| 7'-6" | 158* | 169*⊗ |
| 8'-0" | 156* | 166*⊗ |
| 8'-6" | 152 | 163*⊗ |
| 9'-0" | 150 | 160*⊗ |
| 9'-6" | 147 | 157* |
| 10'-0" | 145 | 154* |
| 10'-6" | 143 | 152 |
| 11'-0" | 140 | 149 |
| 11'-6" | 136 | 147 |
| 12'-0" | 133 | 144 |

**Table 19.3-2**

Maximum Span Length vs. Girder Spacing

* For lateral stability during lifting these girder lengths may require pick-up point locations greater than distance d (girder depth) from the ends of the girder. The designer shall assume that the pick-up points will be at the 1/10 points from the end of the girder and provide extra non-prestressed steel in the top flange, if required, and check the concrete strength near the

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| Length (ft) | 0.0 | 19.0 | 38.0 | 57.0 | 76.0 | 95.0 |
|---|---|---|---|---|---|---|
| Steel Weight | 0 | 400 | 663 | 789 | 778 | 630 |
| Additional Miscellaneous Steel | 0 | 166 | 278 | 336 | 340 | 290 |
| Cast 1 | 0 | 1190 | 1994 | 2413 | 2447 | 2096 |
| Cast 2 | 0 | -29 | -58 | -87 | -116 | -145 |
| Cast 3 | 0 | 25 | 51 | 76 | 102 | 127 |
| Sum of Casts | 0 | 1186 | 1987 | 2402 | 2433 | 2078 |
| Deck & Haunches (Simultaneous Cast) | 0 | 1184 | 1983 | 2396 | 2424 | 2067 |

**Table 24.12-1**
Moments from Deck Placement Analysis (K-ft)

The slight differences in the moments on the last line of Table 24.12-1 (assuming a simultaneous placement of the entire slab) and the sum of the moments due to the three casts are due to the changes in the girder stiffness with each sequential cast. The principle of superposition does not apply directly in the deck-placement analyses, since the girder stiffness changes at each step of the analysis. Although the differences in the moments are small in this example, they can be significantly greater depending on the span configuration. The effects of the deck placement sequence must be considered during design.

In regions of positive flexure, the non-composite girder should be checked for the effect of the maximum accumulated deck-placement moment. This moment at 76 feet from Abutment 1 is computed as:

$$M = 778 + 340 + 2{,}447 = 3{,}565 \text{ kip-ft}$$

This value agrees with the moment at this location shown in Figure 24.12-4.

In addition to the dead load moments during the deck placement, unfactored dead load deflections and reactions can also be investigated similarly during the construction condition.

When investigating reactions during the construction condition, if uplift is found to be present during deck placement, the following options can be considered:

- Rearrange the concrete casts.
- Specify a temporary load over that support.
- Specify a tie-down bearing.
- Perform another staging analysis with zero bearing stiffness at the support experiencing lift-off.

## 24.13 Painting

The final coat of paint on all steel bridges shall be an approved color. Exceptions to this policy may be considered on an individual basis for situations such as scenic river crossings, unique or unusual settings or local community preference. The Region is to submit requests for an exception along with the Structure Survey Report. The colors available for use on steel structures are shown in Chapter 9 - Materials.

Unpainted weathering steel is used on bridges over streams and railroads. All highway grade separation structures require the use of painted steel, since unpainted steel is subject to excessive weathering from salt spray distributed by traffic. On weathering steel bridges, the end 6' of any steel adjacent to either side of an expansion joint and/or hinge is required to have two shop coats of paint. The second coat is to be brown color similar to rusted steel. Do not paint the exterior face of the exterior girders for aesthetic reasons, but paint the hanger bar on the side next to the web. Additional information on painting is presented in Chapter 9 - Materials.

For painted steel plate I-girders utilize a three-coat system defined by the Standard Specification bid item “Painting Epoxy System (Structure)”. For painted tub girders use a two-coat system defined by the SPV “Painting Polysiloxane System (Structure)”, which includes painting of the inside of the tubs.

Paint on bridges affects the slip resistance of bolted connections. Since faying surfaces that are not galvanized are typically blast-cleaned as a minimum, a Class A surface condition should only be used to compute the slip resistance when Class A coatings are applied or when unpainted mill scale is left on the faying surface. Most commercially available primers will qualify as Class B coatings.

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## 29.1 General

Wherever practical, bridge drainage should be carried off the structure along the curb or gutter line and collected with roadway catch basins. Floor drains are not recommended for structures less than 400' long and floor drain spacing is not to exceed 500' on any structure. However, additional floor drains are required on some structures due to flat grades, superelevations and the crest of vertical curves. The drains are spaced according to the criteria as set forth in 29.2, which includes acceptable spread of water measured from gutterline as a function of design speed, design storm frequency and duration of rainfall. Additional drains should not be provided other than what is required by design. Utilizing blockouts in parapets to facilitate drainage is not allowed.

Superelevation on structures often creates drainage problems other than at the low point especially if a reverse curve is involved. Water collects and flows down one gutter and as it starts into the superelevation transition it spreads out over the complete width of roadway at the point of zero cross-slope. From this point the water starts to flow into the opposite gutter. Certain freezing conditions can cause traffic accidents to occur in the flat area between the two transitions. To minimize the problem, locate the floor drain as close to the cross over point as practical. Floor drains are installed as near all joints as practical to prevent gutter flow from passing over and/or through the joints.

The Bureau of Structures recommends the Type “GC” floor drain for new structures. Type "GC" floor drains are gray iron castings that have been tested for hydraulic efficiency. Where hydraulic efficiency or girder flange to edge of deck geometry dictates the use of a different floor drain configuration, BOS recommends the Type “WF” floor drain. Steel fabricated floor drains Type “H” provide an additional 6” of downspout clearance and are retained for maintenance of structures where floor drain size modifications are necessary.

All of the floor drains shown on the Standards have grate inlets. When the longitudinal grade exceeds 1 percent, hydraulic flow testing indicates grates with rectangular longitudinal bars are more efficient than grates having transverse rectangular bars normal to flow. However, grates with bars parallel to the direction of traffic are hazardous to bicyclists and even motorcyclists as bar spacing is increased for hydraulic efficiency. As a result, transverse bars sloped toward the direction of flow are detailed for the cast iron floor drains.

Downspouts are to be fabricated from reinforced thermosetting resin (fiberglass) pipe having a diameter not less than 6” for all new structures. Galvanized standard pipe or reinforced fiberglass material may be used for downspouts when adjusting or rehabilitating existing floor drains. Downspouts are required on all floor drains to prevent water and/or chlorides from getting on the girders, bearings, substructure units, etc. Downspouts should be detailed to extend a minimum of 6” below low prestressed girder bottom flange or 1' below low steel to prevent flange or web corrosion. A downspout collector system is required on all structures over grade separations. Reinforced fiberglass pipe is recommended for all collector systems due to its durability and economy. In the design of collector systems, elimination of unnecessary bends and provision for an adequate number of clean outs is recommended.