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5.1 Factors Governing Bridge Costs

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

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

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

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

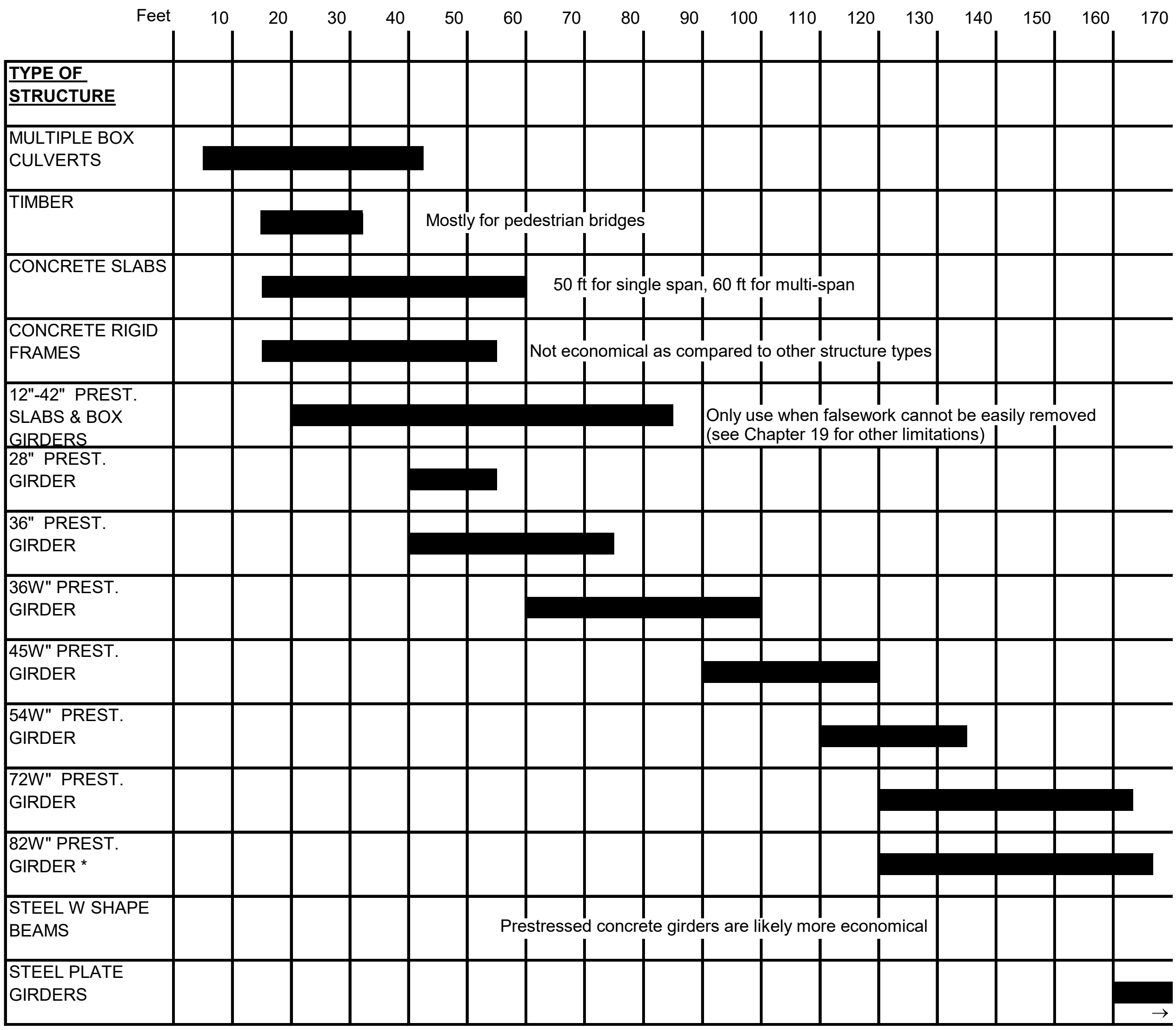
In these reports prestressed girder costs are grouped together because there is a small cost difference between girder sizes. Refer to unit costs. Concrete slab costs are also grouped together for this reason.

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

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

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

5.2 Economic Span Lengths



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

Figure 5.2-1
Economic Span Lengths

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10. List of Drawings

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

11. Title Block

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

12. Professional Seal

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

6.3.2.1.1 Plan Notes for New Bridge Construction

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

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

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

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

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

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

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

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

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

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

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

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

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

6.3.2.1.2 Plan Notes for Bridge Rehabilitation

WisDOT policy item:

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

It is the responsibility of the design engineer to use original structure plans, as-built structure plans, shop drawings, field surveys and structure inspection reports as appropriate when producing rehabilitation structure plans of any type (bridges, retaining walls, box culverts, sign structures, etc.). **Note:** Older Milwaukee bridge plans used a baseline datum of 100.00. Add 580.60 to elevations using this datum. If uncertainty persists after reviewing available documentation, a field visit may be necessary by the design engineer.

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

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

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

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

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

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

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

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

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

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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.

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 detailed in the Site Investigation Report clearly indicate that minimum pile penetration cannot be achieved, preboring should be included as a pay quantity. If there is a potential that preboring may not be necessary, do not include it in the plan documents. Piles which are not prebored into rock must not only meet the 10-foot minimum pile penetration criteria but must also have at least 5 feet of penetration through material with a blow count of at least 7 blows per foot. Piling should be "firmly seated" on rock after placement in prebored holes. The annular space between the cored holes in bedrock and piling should then be filled with concrete. Some sites may require casing during the preboring operation. If casing is required, it should be clearly indicated in the plan documents. Refer to 11.3.1.6 11.3.1.6 for additional information on preboring.

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

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

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

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

11.3.1.2 Pile Spacing

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

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.

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

Table 11.3-6

Economical Evaluation for Deep Foundations with Two Construction Monitoring Methods

11.3.2 Drilled Shafts

11.3.2.1 General

Drilled shafts are generally large diameter, cast-in-place, open ended, cased concrete piles which are designed to carry extremely heavy loads. Drilled shafts can be the most economical foundation alternative at sites where foundation loads are carried to bearing on dense strata or bedrock. They are also cost effective in water crossings with very shallow bedrock, where cofferdams are difficult or expensive to construct, and where high overturning moments must be resisted.

Drilled shafts are installed by removing soil and rock using drilling methods or other excavation techniques and constructing the foundation element in the excavated hole. The excavated hole may be supported using temporary or permanent casing, drilling slurry or other methods. The hole is then filled with a reinforcement cage and cast-in-place concrete. Drilled shafts are non-displacement elements since the soil volume required for the element is physically removed prior to installation. Thus, the effective normal stress adjacent to the pile remains unchanged or is reduced (due to expansion of the soil into the hole before insertion/construction of the load bearing element), and the soil properties and pore water pressure adjacent to the foundation elements are not significantly impacted.

Because drilled shafts do not require a hammer for installation and do not displace the soil, they typically have much less impact on adjacent structures. Depending on the excavation technique used, they can penetrate significant obstructions. Because the method of construction often allows a decrease in the effective stress immediately adjacent to and beneath the tip of the foundation element, the resistance developed will often be less than an equivalently sized driven pile.

Drilled shafts are generally considered fixed to the substructure unit if the reinforcing steel from the shaft is fully developed within the substructure unit.

Drilled shafts vary in diameter from approximately 2.5 to 10 feet. Drilled shafts with diameters greater than 6 feet are generally referred to as piers. Shafts may be designed to transfer load to the bearing stratum through side friction, point-bearing or a combination of both. The drilled shaft may be cased or uncased, depending on the subsurface conditions and depth of bearing.

The minimum drilled shaft spacing shall be 3.0 shaft diameters center-to-center (3D). When drilled shafts are spaced less than 6D, group effects shall be evaluated for possible reductions to axial and lateral resistances. See 11.3.2.3.3 for more information.

Drilled shafts have been used on only a small number of structures in Wisconsin. For unusual site conditions, the use of drilled shafts may be advantageous. Design methodologies for drilled shafts can be found in **LRFD [10.8]** Drilled Shafts and *Drilled Shafts: Construction Procedures and Design Methods*. FHWA Publication NHI-18-024, FHWA GEC 010. 2018.

Strength limit states for drilled shafts are evaluated in the same way as for driven piles. Drivability is not required to be evaluated. The structural resistance of drilled shafts is evaluated in accordance with **LRFD [5.6 and 5.7]**. This includes evaluation of axial resistance, combined axial and flexure, shear and buckling. It is noted that the critical load case for combined axial and flexure may be a load case that results in the minimum axial load or tension.

11.3.2.2 Resistance Factors

Resistance factors for drilled shafts are presented in Table 11.3-7 and are selected based on the method used to determine the nominal (ultimate) resistance capacity of the drilled shaft. The design intent is to adjust the resistance factor based on the reliability of the method used to determine the nominal shaft resistance. As with driven piles, the selection of a geotechnical resistance factor should be based on the intended method of resistance verification in the field. Because of the cost and difficulty associated with testing drilled shafts, much more reliance is placed on static analysis methods.

<table>
<tr><th colspan="4">Condition/Resistance Determination Method</th><th>Resistance Factor</th></tr>
<tr><td rowspan="15">Static Analysis – Used in Design Phase</td><td rowspan="9">Nominal Resistance of Single-Drilled Shaft in Axial Compression, φ_{stat}</td><td>Shaft Resistance in Clay</td><td>Alpha Method</td><td>0.45</td></tr>
<tr><td>Point Resistance in Clay</td><td>Total Stress</td><td>0.40</td></tr>
<tr><td>Shaft Resistance in Sand</td><td>Beta Method</td><td>0.55</td></tr>
<tr><td>Point Resistance in Sand</td><td>O’Neill and Reese</td><td>0.50</td></tr>
<tr><td>Shaft Resistance in IGMs</td><td>O’Neill and Reese</td><td>0.60</td></tr>
<tr><td>Point Resistance in IGMs</td><td>O’Neill and Reese</td><td>0.55</td></tr>
<tr><td rowspan="2">Shaft Resistance in Rock</td><td>Horvath and Kenney
O’Neill and Reese</td><td>0.55</td></tr>
<tr><td>Carter and Kulhawy</td><td>0.50</td></tr>
<tr><td>Point Resistance in Rock</td><td>Canadian Geotech. Soc.
Pressuremeter Method
O’Neill and Reese</td><td>0.50</td></tr>
<tr><td>Block Failure, φ_{bl}</td><td colspan="2">Clay</td><td>0.55</td></tr>
<tr><td rowspan="3">Uplift Resistance of Single-Drilled Shaft, φ_{up}</td><td>Clay</td><td>Alpha Method</td><td>0.35</td></tr>
<tr><td>Sand</td><td>Beta Method</td><td>0.45</td></tr>
<tr><td>Rock</td><td>Horvath and Kenney
Carter and Kulhawy</td><td>0.40</td></tr>
<tr><td>Group Uplift Resistance, φ_{ug}</td><td colspan="2">Sand and Clay</td><td>0.45</td></tr>
<tr><td>Horizontal Geotechnical Resistance of Single Shaft or Pile Group</td><td colspan="2">All Soil Types and Rock</td><td>1.0</td></tr>
</table>

Table 11.3-7

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

For drilled shafts, the base geotechnical resistance factors in Table 11.3-7 assume groups containing two to four shafts, which are slightly redundant. For groups containing at least five elements, the base geotechnical resistance factors in Table 11.3-7 should be increased by 20%.

WisDOT policy item:

When a bent contains at least 5 columns (where each column is supported on a single drilled shaft) the resistance factors in Table 11.3-7 should be increased up to 20 percent for the Strength Limit State.

For piers supported on a single drilled shaft, the resistance factors in Table 11.3-7 should be decreased by 20 percent for the Strength Limit State. Use of single drilled shaft piers requires approval from the Bureau of Structures.

Resistance factors for structural design of drilled shafts are obtained from **LRFD [5.5.4.2]**.

11.3.2.3 Bearing Resistance

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

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

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

11.3.2.3.1 Shaft Resistance

The shaft resistance is estimated by summing the friction developed in each stratum. When drilled shafts are socketed in rock, the shaft resistance that is developed in soil is generally ignored to satisfy strain compatibility. The following analysis methods are typically used to compute the static shaft resistance in soil and rock:

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

- Horvath and Kenny method for rock, as specified in **LRFD [10.8.3.5.4]**

11.3.2.3.2 Point Resistance

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

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

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

11.3.2.3.3 Group Capacity

Group effects for axial and lateral resistances shall be evaluated in accordance with **LRFD [10.8.3.6]** and **LRFD [10.8.3.8]**, respectively. In general, reductions to individual nominal resistances are limited to drilled shafts spaced less than 6D and are based on spacing, soil type, and soil contact.

11.3.2.4 Lateral Load Resistance

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

11.3.2.5 Other Considerations

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

11.3.3 Micropiles

11.3.3.1 General

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

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

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

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

11.3.3.2 Design Guidance

Micropiles shall be designed in conformance with the current *AASHTO LRFD* and in accordance with the WisDOT Bridge Manual. Design guidelines for micropiles are provided in FHWA Publication No. FHWA-NHI-05-039.

11.3.4 Augered Cast-In-Place Piles

11.3.4.1 General

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

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

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

11.3.4.2 Design Guidance

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

11.4 References

1. State of Wisconsin, Department of Transportation. *Standard Specifications for Highway and Structure Construction*.

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

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

4. Armour, T., P. Groneck, J. Keeley and S. Sharma. *Micropile Design and Construction Guidelines, Implementation Manual*. FHWA Publication SA-97-070. June 2000.

5. Chellis, R.D. *Pile Foundations*. McGraw Hill. 1961.

6. Hannigan, P.J., G.G. Goble, G. E. Likins and F. Rausche. *Design and Construction of Driven Pile Foundations – Volumes 1 and 2*. FHWA Publication NHI-05-042. 2006.

7. Kimmerling, R.E., R.C. Bachus, P.W. Mayne, J.A. Scheider and T.E. Zettler. *Geotechnical Engineering Circular No. 6: Shallow Foundations*. FHWA Publication SA-02-054. September 2002.

8. Lagrasse, P.F., J.D. Schall and E.V. Richardson. *Hydraulic Engineering Circular No. 20: Stream Stability at Highway Structures*, 3rd Edition. FHWA Publication NHI 01-002. March 2001.

9. Lagrasse, P.F., L.W. Zevenbergen, J.D. Schall and P.E. Clopper. *Hydraulic Engineering Circular No. 23: Bridge Scour and Stream Instability Countermeasures, Experience, Selection, and Design Guidance*, 2nd Edition. FHWA Publication NHI 01-003. March 2001.

10. Leonards, G.A. *Foundation Engineering*. McGraw Hill. 1962.

11. Michael Baker Jr., Inc. *LRFD for Highway Bridge Substructures and Earth Retaining Structures*. FHWA Publication NHI-05-095. Revised October 2006.

12. *Micropile Design and Construction Reference Manual*. FHWA Publication NHI-05-039. 2005.

13. O'Neil, M.W. and L.C. Reese. *Drilled Shafts: Construction Procedures and Design Methods*. FHWA Publication IF-99-025. August 1999.

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19. *Drilled Shafts: Construction Procedures and Design Methods*. FHWA Publication NHI-18-024, FHWA GEC 010. 2018.

11.5 Design Examples

WisDOT will provide design examples.

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

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12.6 Abutment Drainage and Backfill

This section describes abutment design considerations related to drainage and backfill. The abutment drainage and backfill must be designed and detailed properly to prevent undesirable loads from being applied to the abutment.

12.6.1 Abutment Drainage

Abutment drainage is necessary to prevent hydrostatic pressure and frost pressure. Hydrostatic pressure, including both soil and water, can amount to an equivalent fluid unit weight of soil of 85 pcf. Frost action, which can occur in silty backfill, may result in extremely high pressures. On high abutments, these pressures will produce a very large force which could result in structural damage or abutment movement if not accounted for in the design.

To prevent these additional pressures on abutments, it is necessary to drain away whatever water accumulates behind the body and wings. This is accomplished using a pervious granular fill on the inside face of the abutment. Pipe underdrain must be provided to drain the fill located behind the abutment body and wings. For rehabilitation of structures, provide plan details to replace inadequate underdrain systems.

Past experience indicates that sill abutments are not capable of withstanding hydrostatic pressure on their full height without leaking.

Semi-retaining and full-retaining abutments generally will be overstressed or may slide if subject to large hydrostatic or frost pressures unless accounted for in the design. Therefore, “Pipe Underdrain Wrapped 6-inch” is required behind all abutments. This pipe underdrain is used behind the abutment and outside the abutment to drain the water away. Provide a minimum slope of 0.5% and discharge to suitable drainage (i.e. a storm sewer system or ditch). It is best to place the pipe underdrain along the bottom of footing elevation as per standards. However, if it is not possible to discharge the water to a lower elevation, the pipe underdrain should be placed higher. For bottom of abutments located below the normal water, pipe underdrain should be sloped to discharge a minimum of 1 foot above the normal water elevation.

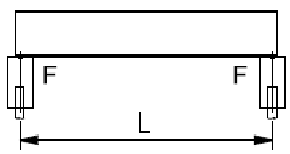
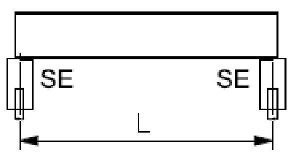
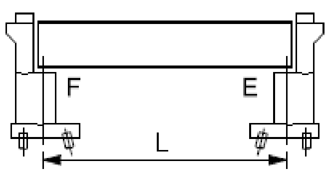
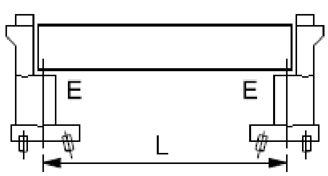
Pipe underdrains and weepholes may discharge water during freezing temperatures. In urban areas, this may create a problem due to the accumulation of flow and ice on sidewalks.

12.6.2 Abutment Backfill Material

All abutments and wings shall utilize “Backfill Structure” to facilitate drainage. See Standard Detail 9.01 – Structure Backfill Limits and Notes – for typical pay limits and plan notes.

12.7 Selection of Standard Abutment Types

From past experience and investigations, the abutment types presented in Figure 12.7-1 are generally most suitable and economical for the given conditions. Although piles are shown for each abutment type, drilled shafts or spread footings may also be utilized depending on the material conditions at the bridge site. The chart in Figure 12.7-1 provides a recommended guide for abutment type selection.

Abutment Arrangements	Superstructures		
	Concrete Slab Spans	Prestressed Girders	Steel Girders
Type A1 (F-F)  	a. L ≤ 150' S ≤ 30° AL ≤ 50'	a. L ≤ 150' S ≤ 15° AL ≤ 50' 28" and 36" only (36W" thru 82W" require SE)	a. L ≤ 150' S ≤ 15° AL ≤ 50'
Type A1 (SE-SE)  	a. L ≤ 300' S ≤ 30° AL > 50'	a. L ≤ 300' S ≤ 40°	a. L ≤ 150' S ≤ 40°
Type A3 (F-E)  	Not used	Single span and (S > 40°)	Single span and (L > 150' or S > 40°)
Type A3 (E-E)  	b. L >300' and S ≤ 30° with rigid piers	L>300' or (S > 40° and multi-span)	Multi-span and (L>150' or S > 40°) with rigid piers

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17.8 Bridge Deck Protective Systems

17.8.1 General

FHWA encourages states that require the use of de-icers to employ bridge deck protective systems. The major problem resulting in bridge deck deterioration is delamination of the concrete near the top mat of the reinforcing steel followed by subsequent spalling of the surface concrete. Research shows that the most prevalent cause of extensive deck deterioration is corrosion of the reinforcing steel due to the intrusion of chlorides into the concrete from repeated de-icer applications during snow and/or ice removal.

Several types of bridge deck protective systems are currently available. Some have been approved by FHWA based on their initial performance. Some of the more common types of protective systems are epoxy coated reinforcing steel, galvanized or stainless steel reinforcing steel, microsilica modified concrete or polymer impregnated concrete, cathodic protection and deck sealers. Epoxy coated reinforcing steel and deck sealers are preferred by WisDOT.

Structures other than box culverts that are designed to carry an earth fill are required to have waterproofing membrane systems on the deck to protect the slab. This includes bridges designed for future grade changes.

17.8.2 Design Guidance

All deck reinforcement bars shall be epoxy coated and the top reinforcing bars shall have a minimum of 2 ½ inches of cover.

All decks shall receive a protective surface treatment. Other locations for protective surface treatment should include: parapet, parapet wing, median, sidewalk and edge of deck/slab and 1’-0” underside of deck/slab when open railings are utilized.

Additional protective systems may be desired to minimize future rehabilitations. One or a combination of systems may be used on large projects such as Mega Projects. Contact the WisDOT Bureau of Structures Design Section for approval and project specific guidance. The following systems are currently being used and should be considered on new structures and deck rehabilitations:

- High Performance Concrete (HPC) – This is typically used within the bridge superstructure (deck, diaphragms, parapets, structural approach slabs, etc.) on urban interchange projects

- Polymer overlays - This system extends the decks service life before rehabilitation is required.

- Stainless steel deck reinforcement – Use of stainless steel in lieu of epoxy bars may be justified for urban interchange projects and complex structures. Savings from reducing the number of rehabilitation projects and user costs can be substantial. Currently, only the enhanced corrosion protection benefits shall be utilized and reinforcement shall be selected per the epoxy coated deck design tables. The use of stainless reinforcing steel shall be approved by Chief Structures Development or Design Engineer and may require a life cycle analysis.

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E_p = Modulus of elasticity of prestressing steel = 28,500 ksi **LRFD [5.4.4.2]**

E_{ct} = Modulus of elasticity of concrete at transfer or time of load application in ksi (see 19.3.3.8)

f_{gcp} = Concrete stress at the center of gravity of prestressing tendons due to the prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment (ksi)

19.3.2.2.2 Time-Dependent Losses

Per **LRFD [5.9.3.3],** an estimate of the long-term losses due to steel relaxation as well as concrete creep and shrinkage on standard precast, pretensioned members shall be taken as:

$$\Delta f_{pLT} = 10.0\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 12.0\gamma_h\gamma_{st} + \Delta f_{pR}$$

Where:

$$\gamma_h = 1.7 - 0.01H$$

$$\gamma_{st} = \frac{5}{(1 + f'_{ci})}$$

f_{pi} = Prestressing steel stress immediately prior to transfer (ksi)

H = Average annual ambient relative humidity in %, taken as 72% in Wisconsin

Δf_{pR} = Relaxation loss estimate taken as 2.4 ksi for low relaxation strands or 10.0 ksi for stress-relieved strands (ksi)

The losses due to elastic shortening must then be added to these time-dependent losses to determine the total losses. For non-standard members with unusual dimensions or built using staged segmental construction, the refined method of **LRFD [5.9.3.4]** shall be used. For prestressed box girders time-dependent losses shall be determined using the refined method of **LRFD [5.9.3.4]**.

19.3.2.2.3 Fabrication Losses

Fabrication losses are not considered by the designer, but they affect the design criteria used during design. Anchorage losses which occur during stressing and seating of the prestressed strands vary between 1% and 4%. Losses due to temperature change in the strands during cold weather prestressing are 6% for a 60°F change. The construction specifications permit a 5% difference in the jack pressure and elongation measurement without any adjustment.

19.3.2.3 Service Load

During service load, the member is subjected to the same loads that are present after prestress transfer and losses occur, in addition to the effects of the prestressed I-girder and prestressed box girder load-carrying behavior described in the next two sections.

19.3.2.3.1 Prestressed I-Girder

In the case of a prestressed I-girder, the dead load of the deck and diaphragms are always carried by the basic girder section on a simple span. At strand release, the girder dead load moments are calculated based on the full girder length. For all other loading stages, the girder dead load moments are based on the span length. This is due to the type of construction used (that is, unshored girders simply spanning from one substructure unit to another for single-span as well as multi-span structures).

The live load plus dynamic load allowance along with any superimposed dead load (curb, parapet or median strip which is placed after the deck concrete has hardened) are carried by the continuous composite section.

WisDOT exception to AASHTO:

The standard pier diaphragm is considered to satisfy the requirements of **LRFD [5.12.3.3.5]** and shall be considered to be fully effective.

In the case of multi-span structures with fully effective diaphragms, the longitudinal distribution of the live load, dynamic load allowance and superimposed dead loads are based on a continuous span structure. This continuity is achieved by:

a. Placing non-prestressed (conventional) reinforcement in the deck area over the interior supports.

b. Casting concrete between and around the abutting ends of adjacent girders to form a diaphragm at the support. Girders shall be in line at interior supports and equal numbers of girders shall be used in adjacent spans. The use of variable numbers of girders between spans requires prior approval by BOS.

It is preferred, but not required, to have a length ratio of two adjacent spans not exceeding 1.5. Short end spans, especially with expansion abutments, can be problematic with regards to having enough dead load to utilize non-anchored laminated elastomeric bearings.

If girder depth changes, the girders would be designed as if the bridge was discontinuous at the shared pier; however, the continuity reinforcement should be designed as if the bridge was being designed continuous at the shared pier. The loads to the shared pier should be determined as if for a continuous bridge (i.e. simple span for non-composite loads and continuous for composite loads).

Bridges may have varying girder spacing between spans. A historically common configuration in Wisconsin for prestressed I-girder superstructures is a four-span bridge with continuous girders in spans 2 & 3 and different (wider) girder spacing in spans 1 & 4 (Note: this configuration is not recommended for new structures). A replacement deck for such bridges would be designed as continuous, although the rating would be as for separate units – single span, two-span and single span.

19.3.2.3.2 Prestressed Box Girder

In the case of prestressed box girders with a thin concrete overlay, the dead load together with the live load and dynamic load allowance are carried by the basic girder section.

When this girder type has a composite section, the dead load of the deck is carried by the basic section and the live load, dynamic load allowance and any superimposed dead loads are carried by the composite section. A composite section shall consist of a reinforced deck, 6" minimum thickness, with composite shear reinforcement extending into the deck.

WisDOT policy item:

The use of prestressed box girders is subject to prior-approval by the Bureau of Structures. These structures are currently limited to the following requirements:

- Single spans
- Composite section details (design and rating based on non-composite section)
- 30 degree maximum skew
- AADT < 3,500 on non-NHS roadways

Variations to these requirements require approval by the Bureau of Structures.

19.3.2.4 Factored Flexural Resistance

At the final stage, the factored flexural resistance of the composite section is considered. Since the member is designed on a service load basis, it must be checked for its factored flexural resistance at the Strength I limit state. See section 17.2.3 for a discussion on limit states.

The need for both service load and strength computations lies with the radical change in a member's behavior when cracks form. Prior to cracking, the gross area of the member is effective. As a crack develops, all the tension in the concrete is picked up by the reinforcement. If the percentage of reinforcement is small, there is very little added capacity between cracking and failure.

19.3.2.5 Fatigue Limit State

At the final stage, the member is checked for the Fatigue I limit state. See section 17.2.3 for a discussion on limit states. Allowable compressive stresses in the concrete and tensile stresses in the non-prestressed reinforcement are checked.

19.3.3 Design Procedure

The intent of this section is to provide the designer with a general outline of steps for the design of pretensioned members. Sections of interest during design include, but are not limited to, the following locations:

- 10^{th} points
- Hold-down points
- Regions where the prestress force changes (consider the effects of transfer and development lengths, as well as the effects of debonded strands)
- Critical section(s) for shear

The designer must consider the amount of prestress force at each design section, taking into account the transfer length and development length, if appropriate.

19.3.3.1 Prestressed I-Girder Member Spacing

A trial prestressed I-girder arrangement is made by using Table 19.3-1 and Table 19.3-2 as a guide. An ideal spacing results in equal strands for interior and exterior girders, together with an optimum slab thickness. Current practice is to use a minimum haunch of (1-1/4" plus deck cross slope times one-half top flange width) for section property calculations and then use a 3" average haunch for concrete preliminary quantity calculations. After preliminary design this value should be revised as needed as outlined in 19.3.4. The maximum slab overhang dimensions are detailed in 17.6.2.

For prestressed I-girder bridges, other than pedestrian or other unusual structures, four or more girders shall be used.

19.3.3.2 Prestressed Box Girder Member Spacing

The prestressed box girder is used in an adjacent multi-beam system only. Precast units are placed side by side and locked (post-tensioned) together. The span length, desired roadway width and live loading control the size of the member.

When selecting a 3' wide section vs. 4' wide section, do not mix 3' wide and 4' wide sections across the width of the bridge. Examine the roadway width produced by using all 3' wide sections or all 4' wide sections and choose the system that is the closest to but greater than the required roadway width. While 3' wide sections may produce a slightly narrower roadway width 4' wide sections are still preferred since they require fewer sections. Verify the required roadway width is possible when considerations are made for the roadway cross-slope. Table 19.3-3 states the approximate span limitations for each section depth. Coordinate roadway width with roadway designers and consider some variability. See the Standards for prestressed box girder details.

19.3.3.3 Dead Load

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

The dead load moments and shears due to the girder and concrete deck are computed for simple spans. When superimposed dead loads are considered, the superimposed dead load moments are based on continuous spans.

A superimposed dead load of 20 psf is to be included in all designs which account for a possible future concrete overlay wearing surface. The future wearing surface shall be applied between the faces of curbs or parapets and shall be equally distributed among all the girders in the cross section.

For a cross section without a sidewalk, any curb or parapet dead load is distributed equally to all girders.

For a cross section with a sidewalk and barrier on the overhang, sidewalk and barrier dead loads shall be applied to the exterior girder by the lever rule. These loads shall also be applied to the interior girder by dividing the weight equally among all the girders. A more detailed discussion of dead load distribution can be found in 17.2.8.

19.3.3.4 Live Load

The HL-93 live load shall be used for all new bridges. Refer to section 17.2.4.2 for a detailed description of the HL-93 live load, including the design truck, design tandem, design lane, and double truck.

19.3.3.5 Live Load Distribution

The live load distribution factors shall be computed as specified in **LRFD [4.6.2.2].** Table 17.2-7 summarizes the equations required for prestressed I-girders. The moment and shear distribution factors for prestressed I-girders are determined using equations that consider girder spacing, span length, deck thickness, the number of girders, skew and the longitudinal stiffness parameter. See the WisDOT policy item for live load distribution factors for prestressed box girders.

Separate shear and moment distribution factors are computed for interior and exterior girders. The applicability ranges of the distribution factors shall also be considered. If the applicability ranges are not satisfied, then conservative assumptions must be made based on sound engineering judgment.

WisDOT policy item:

The typical cross section for prestressed box girders shall be type “g” as illustrated in **LRFD [Table 4.6.2.2.1-1]**.

For prestressed box girders, the St. Venant torsional inertia, J, may be calculated as closed thin-walled sections for sections with voids, and as solid sections for sections without voids in accordance with **LRFD [C4.6.2.2.1]**.

See 17.2.8 for additional information regarding live load distribution.

19.3.3.6 Dynamic Load Allowance

The dynamic load allowance, IM, is given by **LRFD [3.6.2]**. Dynamic load allowance equals 33% for all live load limit states except the fatigue limit state and is not applied to pedestrian loads or the lane load portion of the HL-93 live load. See 17.2.4.3 for further information regarding dynamic load allowance.

19.3.3.7 Prestressed I-Girder Deck Design

The design of concrete decks on prestressed I-girders is based on **LRFD [4.6.2.1]**. Moments from truck wheel loads are distributed over a width of deck which spans perpendicular to the girders. This width is known as the distribution width and is given by **LRFD [Table 4.6.2.1.3-1]**. See 17.5 for further information regarding deck design.

19.3.3.8 Composite Section

The effective flange width is the width of the deck slab that is to be taken as effective in composite action for determining resistance for all limit states. The effective flange width, in accordance with **LRFD [4.6.2.6]**, is equal to the tributary width of the girder for interior girders. For exterior girders, it is equal to one half the effective flange width of the adjacent interior girder plus the overhang width. The effective flange width shall be determined for both interior and exterior beams.

Since the deck concrete has a lower strength than the girder concrete, it also has a lower modulus of elasticity. Therefore, when computing composite section properties, the effective flange width (as stated above) must be reduced by the ratio of the modulus of elasticity of the deck concrete divided by the modulus of elasticity of the girder concrete.

WisDOT exception to AASHTO:

WisDOT uses the formulas shown below to determine E_c for prestressed girder design. For 6 ksi girder concrete, E_c is 5,500 ksi, and for 4 ksi deck concrete, E_c is 4,125 ksi. The E_c value of 5,500 ksi for 6 ksi girder concrete strength was determined from deflection studies. These equations are used in place of those presented in **LRFD [5.4.2.4]** for the following calculations: strength, section properties, and deflections due to externally applied dead and live loads.

For slab concrete strength other than 4 ksi, E_c is calculated from the following formula:

$$E_c = \frac{4{,}125\sqrt{f'_c}}{\sqrt{4}} \text{ (ksi)}$$

For girder concrete strengths other than 6 ksi, E_c is calculated from the following formula:

$$E_c = \frac{5{,}500\sqrt{f'_c}}{\sqrt{6}} \text{ (ksi)}$$

WisDOT policy item:

WisDOT uses the equation presented in **LRFD [5.4.2.4]** (and shown below) to calculate the modulus of elasticity at the time of release using the specified value of f'ci. This value of E_i is used for loss calculations and for girder camber due to prestress forces and girder self-weight.

$$E_c = 33{,}000 \cdot K_1 \cdot w_c^{1.5}\sqrt{f'_{ci}}$$

Where:

K_1	=	Correction factor for source of aggregate, use 1.0 unless previously approved by BOS.
w_c	=	Unit weight of concrete, 0.150 (kcf)
f'_{ci}	=	Specified compressive strength of concrete at the time of release (ksi)

19.3.3.9 Design Stress

In many cases, stress at the Service III limit state in the bottom fiber at or near midspan after losses will control the flexural design. Determine a trial strand pattern for this condition and proceed with the flexural design, adjusting the strand pattern if necessary.

The design stress is the sum of the Service III limit state bottom fiber stresses due to non-composite dead load on the basic girder section, plus live load, dynamic load allowance and superimposed dead load on the composite section, as follows:

$$f_{des} = \frac{M_{d(nc)}}{S_{b(nc)}} + \frac{M_{d(c)} + M_{(LL+IM)}}{S_{b(c)}}$$

Where:

f_{des}	=	Service III design stress at section (ksi)
$M_{d(nc)}$	=	Service III non-composite dead load moment at section (k-in)
$M_{d(c)}$	=	Service III superimposed dead load moment at section (k-in)
$M_{(LL+IM)}$	=	Service III live load plus dynamic load allowance moment at section (k-in)

$S_{b(nc)}$ = Non-composite section modulus for bottom of basic beam (in^3)

$S_{b(c)}$ = Composite section modulus for bottom of basic beam (in^3)

The point of maximum stress is generally 0.5 of the span for both end and intermediate spans. But for longer spans (over 100'), the 0.4 point of the end span may control and should be checked.

19.3.3.10 Prestress Force

With f_{des} known, compute the required effective stress in the prestressing steel after losses, f_{pe}, needed to counteract all the design stress except an amount of tension equal to the tensile stress limit listed in **LRFD [Table 5.9.2.3.2b-1]**. The top of the girder is subjected to severe corrosion conditions and the bottom of the girder is subjected to moderate exposure. The Service III tensile stress at the bottom fiber after losses for pretensioned concrete shall not exceed $0.19\lambda\sqrt{f'_c}$ (or 0.6 ksi) ; where λ = concrete density modification factor **LRFD [5.4.2.8]**, and has a value of 1.0 for normal weight concrete. Therefore:

$$f_{pe} = f_{des} - \min(0.19\sqrt{f'_c} \text{ or } 0.6 \text{ ksi})$$

Note: A conservative approach used in hand calculations is to assume that the allowable tensile stress equals zero.

Applying the theory discussed in 19.2:

$$f_{pe} = \frac{P_{pe}}{A}\left(1 + \frac{ey}{r^2}\right)$$

Where:

P_{pe} = Effective prestress force after losses (kips)

A = Basic beam area (in^2)

e = Eccentricity of prestressing strands with respect to the centroid of the basic beam at section (in)

r = $\sqrt{\frac{I}{A}}$ of the basic beam (in)

For prestressed box girders, assume an e and apply this to the above equation to determine P_{pe} and the approximate number of strands. Then a trial strand pattern is established using the Standard Details as a guide, and a check is made on the assumed eccentricity. For prestressed

I-girders, f_{pe} is solved for several predetermined patterns and is tabulated in the Standard Details.

Present practice is to detail all spans of equal length with the same number of strands, unless a span requires more than three additional strands. In this case, the different strand arrangements are detailed along with a plan note stating: "The manufacturer may furnish all girders with the greater number of strands."

19.3.3.11 Service Limit State

Several checks need to be performed at the service limit state. Refer to the previous narrative in 19.3.3 for sections to be investigated and section 17.2.3.2 for discussion on the service limit state. Note that Service I limit state is used when checking compressive stresses and Service III limit state is used when checking tensile stresses.

The following should be verified by the engineer:

- Verify that the Service III tensile stress due to beam self-weight and prestress applied to the basic beam at transfer does not exceed the limits presented in **LRFD [Table 5.9.2.3.1b-1]**, which depend upon whether or not the strands are bonded and satisfy stress requirements. This will generally control at the top of the beam near the beam ends where the dead load moment approaches zero and is not able to counter the tensile stress at the top of the beam induced by the prestress force. When the calculated tensile stress exceeds the stress limits, the strand pattern must be modified by draping or partially debonding the strand configuration.

- Verify that the Service I compressive stress due to beam self-weight and prestress applied to the basic beam at transfer does not exceed 0.65 f'_{ci}, as presented in **LRFD [5.9.2.3.1a]**. This will generally control at the bottom of the beam near the beam ends or at the hold-down point if using draped strands.

- Verify that the Service III tensile stress due to all dead and live loads applied to the appropriate sections after losses does not exceed the limits presented in **LRFD [Table 5.9.2.3.2b-1]**. No tensile stress shall be permitted for unbonded strands. The tensile stress of bonded strands shall not exceed $0.19\lambda\sqrt{f'_c}$ (or 0.6 ksi) as all strands shall be considered to be in moderate corrosive conditions. This will generally control at the bottom of the beam near midspan and at the top of the continuous end of the beam. The value of λ is 1.0 for normal weight concrete **LRFD [5.4.2.8]**.

- Verify that the Service I compressive stress due to all dead and live loads applied to the appropriate sections after losses does not exceed the limits presented in **LRFD [Table 5.9.2.3.2a-1]**. Two checks need to be made for girder bridges. The compressive stress due to the sum of effective prestress and permanent loads shall not exceed 0.45 f'_c (ksi). The compressive stress due to the sum of effective prestress, permanent loads and transient loads shall not exceed $0.60\phi_w f'_c$ (ksi). The term ϕ_w, a reduction factor applied to thin-walled box girders, shall be 1.0 for WisDOT standard girders.

- Verify that Fatigue I compressive stress due to fatigue live load and one-half the sum of effective prestress and permanent loads does not exceed 0.40 f'_c (ksi) **LRFD [5.5.3.1]**.
- Verify that the Service I compressive stress at the top of the deck due to all dead and live loads applied to the appropriate sections after losses does not exceed 0.40 f'_c.

WisDOT policy item:

The top of the prestressed I-girders at interior supports shall be designed as reinforced concrete members at the strength limit state in accordance with **LRFD [5.12.3.3.6]**. In this case, the stress limits for the service limit state shall not apply to this region of the precast girder.

19.3.3.12 Raised, Draped or Partially Debonded Strands

When straight strands are bonded for the full length of a prestressed girder, the tensile and compressive stresses near the ends of the girder will likely exceed the allowable service limit state stresses. This occurs because the strand pattern is designed for stresses at or near midspan, where the dead load moment is highest and best able to balance the effects of the prestress. Near the ends of the girder this dead load moment approaches zero and is less able to balance the prestress force. This results in tensile stresses in the top of the girder and compressive stresses in the bottom of the girder. The allowable initial tensile and compressive stresses are presented in the first two bullet points of 19.3.3.11. These stresses are a function of f'_{ci}, the compressive strength of concrete at the time of prestress force transfer. Transfer and development lengths should be considered when checking stresses near the ends of the girder.

The designer should start with a straight (raised), fully bonded strand pattern. If this overstresses the girder near the ends, the following methods shall be utilized to bring the girder within the allowable stresses. These methods are listed in order of preference and discussed in the following sections:

1. Use raised strand pattern (If excessive top flange reinforcement or if four or more additional strands versus a draped strand pattern are required, consider the draped strand alternative)

2. Use draped strand pattern

3. Use partially debonded strand pattern (to be used sparingly)

Only show one strand pattern per span (i.e. Do not show both raised and draped span alternatives for a given span).

A different girder spacing may need to be selected. It is often more economical to add an extra girder line than to maximize the number of strands and use debonding.

Prestressed box girders strands are to be straight, bonded, and located as shown in the Standard Details.

19.3.3.12.1 Raised Strand Patterns

Some of the standard strand patterns listed in the Standard Details show a raised strand pattern. Generally strands are placed so that the center of gravity of the strand pattern is as close as possible to the bottom of the girder. With a raised strand pattern, the center of gravity of the strand pattern is raised slightly and is a constant distance from the bottom of the girder for its entire length. Present practice is to show a standard raised arrangement as a preferred alternate to draping for short spans. For longer spans, debonding at the ends of the strands is an alternate (see 19.3.3.12.3). Use 0.6" strands for all raised patterns.

19.3.3.12.2 Draped Strand Patterns

Draping some of the strands is another available method to decrease stresses from prestress at the ends of the I-beam where the stress due to applied loads are minimum.

The typical strand profile for this technique is shown in Figure 19.3-1.

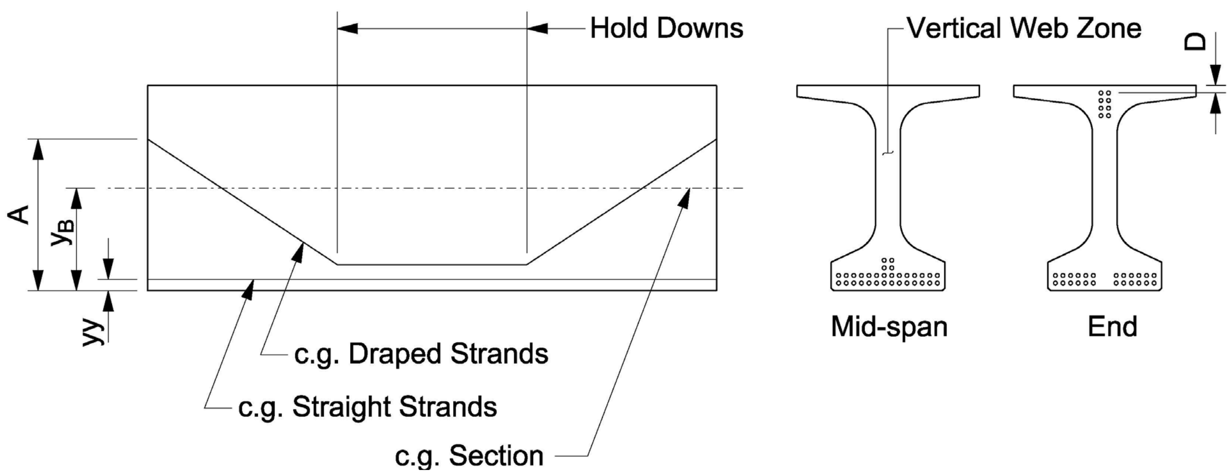

Figure 19.3-1
Typical Draped Strand Profile

Note that all the strands that lie within the "vertical web zone" of the mid-span arrangement are used in the draped group.

The engineer should show only one strand size for the draped pattern on the plans. Use only 0.5" strands for the draped pattern on 28" and 36" prestressed I-girders and 0.6" strands for all raised (straight) patterns for these shapes. Use 0.6" strands, only, for 36W", 45W", 54W", 72W" and 82W" prestressed I-girders. See Chapter 40 standards for 45", 54" and 70" prestressed I-girders.

Hold-down points for draped strands are located approximately between the 1/3 point and the 4/10 point from each end of the girder. The Standard Details, Prestressed Girder Details, show B values at the 1/4 point of the girder. On the plan sheets provide values for B_{min} and B_{max} as determined by the formulas shown on the Standards.

The maximum slope specified for draped strands is 12%. This limit is determined from the safe uplift load per strand of commercially available strand restraining devices used for hold-downs. The minimum distance, D, allowed from center of strands to top of flange is 2”. For most designs, the maximum allowable slope of 12% will determine the location of the draped strands. Using a maximum slope will also have a positive effect on shear forces.

Initial girder stresses are checked at the end of the transfer length, which is located 60 strand diameters from the girder end. The transfer length is the embedment length required to develop f_{pe}, the effective prestressing steel stress (ksi) after losses. The prestressing steel stress varies linearly from 0.0 to f_{pe} along the transfer length.

The longer full development length of the strand is required to reach the larger prestressing steel stress at nominal resistance, f_{ps} (ksi). The strand stress is assumed to increase linearly from f_{pe} to f_{ps} over the distance between the transfer length and development length.

Per **LRFD [5.9.4.3.2]**, the development length is:

$$\ell_d \geq \kappa\left(f_{ps} - \frac{2}{3}f_{pe}\right)d_b$$

Where:

d_b	=	Nominal strand diameter (in)
κ	=	1.0 for members with a depth less than or equal to 24”, and 1.6 for members with a depth of greater than 24”

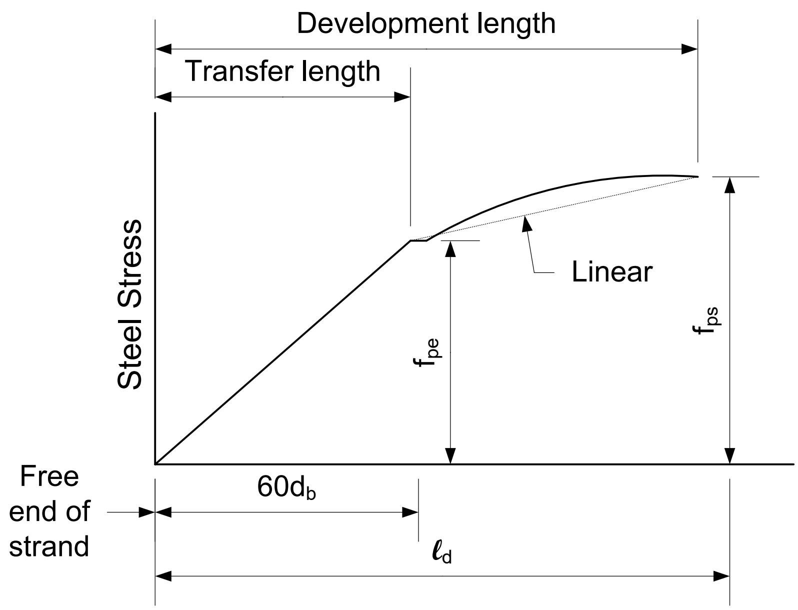

Figure 19.3-2
Transfer and Development Length

19.3.3.12.3 Partially Debonded Strand Patterns

The designer may use debonded strands if a raised or draped strand configuration fails to meet the allowable service stresses. The designer should exercise caution when using debonded strands as this may not result in the most economical design. Partially debonded strands are fabricated by wrapping sleeves around individual strands for a specified length from the ends of the girder, rendering the bond between the strand and the girder concrete ineffective for the wrapped, or shielded, length.

Bond breakers should only be applied to interior strands as girder cracking has occurred when they were applied to exterior strands. In computing bond breaker lengths, consideration is given to the theoretical stresses at the ends of the girder. These stresses are due entirely to prestress. As a result, the designer may compute a stress reduction based on certain strands having bond breakers. This reduction can be applied along the length of the debonded strands.

Partially debonded strands must adhere to the requirements listed in **LRFD [5.9.4.3.3]**. The list of requirements is as follows:

- The development length of partially debonded strands shall be calculated in accordance with **LRFD [5.9.4.3.2]** with $\kappa = 2.0$.

- The number of debonded strands shall not exceed 25% of the total number of strands.

- The number of debonded strands in any horizontal row shall not exceed 40% of the strands in that row.
- The length of debonding shall be such that all limit states are satisfied with consideration of the total developed resistance (transfer and development length) at any section being investigated.
- Not more than 40% of the debonded strands, or four strands, whichever is greater, shall have debonding terminated at any section.
- The strand pattern shall be symmetrical about the vertical axis of the girder. The consideration of symmetry shall include not only the strands being debonded but their debonded length as well, with the goal of keeping the center of gravity of the prestress force at the vertical centerline of the girder at any section. If the center of gravity of the prestress force deviates from the vertical centerline of the girder, the girder will twist, which is undesirable.
- Exterior strands in each horizontal row shall be fully bonded for crack control purposes.

19.3.3.13 Strength Limit State

The design factored positive moment is determined using the following equation:

$$M_u = 1.25DC + 1.50DW + 1.75(LL + IM)$$

The Strength I limit state is applied to both simple and continuous span structures. See 17.2.4 for further information regarding loads and load combinations.

19.3.3.13.1 Factored Flexural Resistance

The nominal flexural resistance assuming rectangular behavior is given by **LRFD [5.6.3.2.3]** and **LRFD [5.6.3.2.2]**.

The section will act as a rectangular section as long as the depth of the equivalent stress block, a, is less than or equal to the depth of the compression flange (the structural deck thickness). Per **LRFD [5.6.3.2.2]**:

$$a = c\beta_1$$

Where:

c	=	Distance from extreme compression fiber to the neutral axis assuming the tendon prestressing steel has yielded (in)
β_1	=	Stress block factor **LRFD [5.6.2.2]**

By neglecting the area of mild compression and tension reinforcement, the equation presented in **LRFD [5.7.3.1.1]** for rectangular section behavior reduces to:

$$c = \frac{A_{ps} f_{pu}}{\alpha_1 f'_c \beta_1 b + k A_{ps} \dfrac{f_{pu}}{d_p}}$$

Where:

A_{ps}	=	Area of prestressing steel (in^2)
f_{pu}	=	Specified tensile strength of prestressing steel (ksi)
f'_c	=	Compressive strength of the flange ($f'_{c(deck)}$ for rectangular section) (ksi)
b	=	Width of compression flange (in)
k	=	0.28 for low relaxation strand per **LRFD [C5.6.3.1.1]**
d_p	=	Distance from extreme compression fiber to the centroid of the prestressing tendons (in)
α_1	=	Stress block factor; equals 0.85 (for $f'_C \leq 10.0$ ksi) **LRFD [5.6.2.2]**

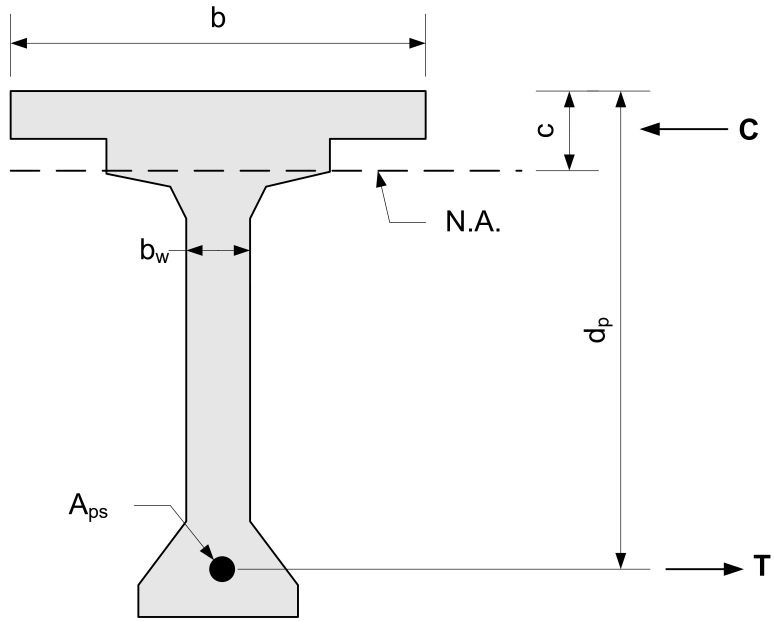

Figure 19.3-3
Depth to Neutral Axis, c

Verify that rectangular section behavior is allowed by checking that the depth of the equivalent stress block, a, is less than or equal to the structural deck thickness. If it is not, then T-section behavior provisions should be followed. If the T-section provisions are used, the compression block will be composed of two different materials with different compressive strengths. In this situation, **LRFD [C5.6.2.2]** recommends using β_1 and α_1 corresponding to the lower f'_c. The following equation for c shall be used for T-section behavior: **LRFD [5.6.3.1.1]**

$$c = \frac{A_{ps} f_{pu} - \alpha_1 f'_c (b - b_w) h_f}{\alpha_1 f'_c \beta_1 b_w + k A_{ps} \dfrac{f_{pu}}{d_p}}$$

Where:

b_w	=	Width of web (in) – use the top flange width if the compression block does not extend below the haunch.
h_f	=	Depth of compression flange (in)

The factored flexural resistance presented in **LRFD [5.6.3.2.2]** is simplified by neglecting the area of mild compression and tension reinforcement. Furthermore, if rectangular section behavior is allowed, then b_w = b, where b_w is the web width as shown in Figure 19.3-3. The equation then reduces to:

$$M_r = \phi A_{ps} f_{ps} \left(d_p - \frac{a}{2} \right)$$

Where:

M_r	=	Factored flexural resistance (kip-in)
ϕ	=	Resistance factor
f_{ps}	=	Average stress in prestressing steel at nominal bending resistance (refer to **LRFD [5.6.3.1.1]**) (ksi)

If the T-section provisions must be used, the factored moment resistance equation is then:

$$M_r = \phi A_{ps} f_{ps} \left(d_p - \frac{a}{2} \right) + \alpha_1 \phi f'_c (b - b_w) h_f \left(\frac{a}{2} - \frac{h_f}{2} \right)$$

Where:

h_f	=	Depth of compression flange with width, b (in)

The engineer must then verify that M_r is greater than or equal to M_u.

WisDOT exception to AASHTO:

WisDOT standard prestressed I-girders and strand patterns are tension-controlled. The ε_t check, as specified in **LRFD [5.6.2.1]**, is not required when the standard girders and strand patterns are used, and $\phi = 1$.

19.3.3.13.2 Minimum Reinforcement

Per **LRFD [5.6.3.3]**, the minimum amount of prestressed reinforcement provided shall be adequate to develop a M_r at least equal to the lesser of M_{cr}, or $1.33M_u$.

M_{cr} is the cracking moment, and is given by:

$$M_{cr} = \gamma_3 \left[S_c \left(\gamma_1 f_r + \gamma_2 f_{cpe} \right) - 12M_{dnc} \left[(S_c/S_{nc}) - 1 \right] \right]$$

Where:

S_c	=	Section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (in^3)
f_r	=	Modulus of rupture (ksi)
f_{cpe}	=	Compressive stress in concrete due to effective prestress forces only (after losses) at extreme fiber of section where tensile stress is caused by externally applied loads (ksi)
M_{dnc}	=	Total unfactored dead load moment acting on the basic beam (k-ft)
S_{nc}	=	Section modulus for the extreme fiber of the basic beam where tensile stress is caused by externally applied loads (in^3)
γ_1	=	1.6 flexural cracking variability factor
γ_2	=	1.1 prestress variability factor
γ_3	=	1.0 for prestressed concrete structures

Per **LRFD [5.4.2.6]**, the modulus of rupture for normal weight concrete is given by:

$f_r = 0.24\lambda\sqrt{f'_c}$; where λ = conc. density modification factor **LRFD [5.4.2.8]**, and has a value of 1.0 for normal weight conc.

19.3.3.14 Non-prestressed Reinforcement

Non-prestressed reinforcement consists of bar steel reinforcement used in the conventional manner. It is placed longitudinally along the top of the member to carry any tension which may develop after transfer of prestress. The designer should completely detail all rebar layouts including stirrups.

The amount of reinforcement is that which is sufficient to resist the total tension force in the concrete based on the assumption of an uncracked section.

For draped designs, the control is at the hold-down point of the girder. At the hold-down point, the initial prestress is acting together with the girder dead load stress. This is where tension due to prestress is still maximum and compression due to girder dead load is decreasing.

For non-draped designs, the control is at the end of the member where prestress tension exists but dead load stress does not.

Note that a minimum amount of reinforcement is specified in the Standards. This is intended to help prevent serious damage due to unforeseeable causes like improper handling or storing.

19.3.3.15 Horizontal Shear Reinforcement

The horizontal shear reinforcement resists the Strength I limit state horizontal shear that develops at the interface of the slab and girder in a composite section. The dead load used to calculate the horizontal shear should only consider the DC and DW dead loads that act on the composite section. See 17.2.4 for further information regarding the treatment of dead loads and load combinations.

$$V_u = 1.25DC + 1.50DW + 1.75(LL + IM)$$

$$V_{ni} \geq V_{ui} / \phi$$

Where:

V_u	=	Maximum strength limit state vertical shear (kips)
V_{ui}	=	Strength limit state horizontal shear at the girder/slab interface (kips)
V_{ni}	=	Nominal interface shear resistance (kips)
ϕ	=	0.90 per **LRFD [5.5.4.2]**

The shear stress at the interface between the slab and the girder is given by:

$$v_{ui} = \frac{V_u}{b_{vi} d_v}$$

Where:

v_{ui} = Factored shear stress at the slab/girder interface (ksi)

b_{vi} = Interface width to be considered in shear transfer (in)

d_v = Distance between the centroid of the girder tension steel and the mid-thickness of the slab (in)

The factored horizontal interface shear shall then be determined as:

$$V_{ui} = 12 v_{ui} b_{vi}$$

The nominal interface shear resistance shall be taken as:

$$V_{ni} = cA_{cv} + \mu\left[A_{vf} f_y + P_c\right]$$

Where:

A_{cv} = Concrete area considered to be engaged in interface shear transfer. This value shall be set equal to $12b_{vi}$ (ksi)

c = Cohesion factor specified in **LRFD [5.7.4.4]**. This value shall be taken as 0.28 ksi for WisDOT standard girders with a cast-in-place deck

μ = Friction factor specified in **LRFD [5.7.4.4]**. This value shall be taken as 1.0 for WisDOT standard girders with a cast-in-place deck (dim.)

A_{vf} = Area of interface shear reinforcement crossing the shear plan within the area A_{cv} (in^2)

f_y = Yield stress of shear interface reinforcement not to exceed 60 (ksi)

P_c = Permanent net compressive force normal to the shear plane (kips)

P_c shall include the weight of the deck, haunch, parapets, and future wearing surface. A conservative assumption that may be considered is to set $P_c = 0.0$.

The nominal interface shear resistance, V_{ni}, shall not exceed the lesser of:

$$V_{ni} \leq K_1 f'_c A_{cv} \quad \text{or} \quad V_{ni} \leq K_2 A_{cv}$$

Where:

K_1 = Fraction of concrete strength available to resist interface shear as specified in **LRFD [5.7.4.4]**. This value shall be taken as 0.3 for WisDOT standard girders with a cast-in-place deck (dim.)

K_2 = Limiting interface shear resistance as specified in **LRFD [5.7.4.4]**. This value shall be taken as 1.8 ksi for WisDOT standard girders with a cast-in-place deck

WisDOT policy item:

The stirrups that extend into the deck slab presented on the Standards are considered adequate to satisfy the minimum reinforcement requirements of **LRFD [5.7.4.2]**

19.3.3.16 Web Shear Reinforcement

Web shear reinforcement consists of placing conventional reinforcement perpendicular to the axis of the girder.

WisDOT policy item:

Web shear reinforcement shall be designed by **LRFD [5.7.3.4.2]** (General Procedure) using the Strength I limit state for WisDOT standard girders.

WisDOT prefers girders with spacing symmetrical about the midspan in order to simplify design and fabrication. The designer is encouraged to simplify the stirrup arrangement as much as possible. For vertical stirrups, the required area of web shear reinforcement is given by the following equation:

$$A_v \geq \frac{(V_n - V_c)s}{f_y d_v \cot\theta} \quad \text{(or } 0.0316\lambda\sqrt{f'_c}\frac{b_v s}{f_y} \text{ minimum , } \textbf{LRFD [5.7.2.5]}\text{)}$$

Where:

A_v	=	Area of transverse reinforcement within distance, s (in^2)
V_n	=	Nominal shear resistance (kips)
V_c	=	Nominal shear resistance of the concrete (kips)
s	=	Spacing of transverse reinforcement (in)
f_y	=	Specified minimum yield strength of transverse reinforcement (ksi)
d_v	=	Effective shear depth as determined in **LRFD [5.7.2.8]** (in)
θ	=	Angle of inclination of diagonal compressive stresses as determined in **LRFD 5.7.3.4** (degrees)
b_v	=	Minimum web width within the depth d_v, (in)
λ	=	Concrete density modification factor; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

θ shall be taken as follows:

$$\theta = 29 + 3500\varepsilon_s$$

Where:

ε_s = Net longitudinal tensile strain in the section at the centroid of the tension reinforcement.

$$= \frac{\left(\frac{|M_u|}{d_v}+0.5N_u+|V_u-V_p|-A_{ps}f_{po}\right)}{E_sA_s+E_pA_{ps}}$$

Where:

$|M_u|$ = Absolute value of the factored moment at the section, not taken less than $|V_u-V_p|d_v$ (kip-in.)

N_u = Factored axial force, taken as positive if tensile and negative if compression (kip)

V_p = Component of prestressing force in the direction of the shear force; positive if resisting the applied shear (kip)

A_{ps} = Area of prestressing steel on the flexural tension side of the member (in^2).

A_s = Area of nonprestressing steel on the flexural tension side of the member (in^2).

f_{po} = A parameter taken as modulus of elasticity of prestressing steel multiplied by the locked-in difference in strain between the prestressing steel and the surrounding concrete (ksi).

$$V_u = 1.25DC + 1.5DW + 1.75(LL + IM)$$
$$V_n = V_u / \phi$$

Where:

V_u = Strength I Limit State shear force (kips)

ϕ = 0.90 per **LRFD [5.5.4.2]**

See 17.2 for further iformation regarding load combinations.

Per **LRFD [5.7.3.3]**, determine V_c as given by:

$$V_c = 0.0316\beta\lambda\sqrt{f'_c}\,b_vd_v$$

Where:

β = Factor indicating ability of diagonally cracked concrete to transmit tension and shear. **LRFD [5.7.3.4]**

λ = Concrete density modification factor; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

Where:

$\beta = \dfrac{4.8}{(1+750\varepsilon_s)}$ (For sections containing at least the minimum amount of transverse reinforcement specified in **LRFD [5.7.2.5]**)

WisDOT policy item:

Based on past performance, for prestressed I-girders the upper limit for web reinforcement spacing, s_{max}, per **LRFD [5.7.2.6]** will be reduced to 18 inches.

When determining shear reinforcement, spacing requirements as determined by analysis at 1/10th points, for example, should be carried-out to the next 1/10th point. As an illustration, spacing requirements for the 1/10th point should be carried out to very close to the 2/10th point, as the engineer, without a more refined analysis, does not know what the spacing requirements would be at the 0.19 point. For the relatively small price of stirrups, don't shortchange the shear capacity of the prestressed girder.

The web reinforcement spacing shall not exceed the maximum permitted spacing determined as:

- If $\upsilon_u < 0.125f'_c$, then s_{max} = $0.8d_v \leq 18"$
- If $\upsilon_u \geq 0.125f'_c$, then s_{max} = $0.4d_v \leq 12"$

Where:

$\upsilon_u = \dfrac{V_u - \phi V_p}{\phi b_v d_v}$ per **LRFD [5.7.2.8]**.

The nominal shear resistance, $V_c + V_s$, is limited by the following:

$$V_c + \frac{A_v f_y d_v \cot\theta}{s} \leq 0.25f'_c b_v d_v$$

Reinforcement in the form of vertical stirrups is required at the extreme ends of the girder. The stirrups are designed to resist 4% of the total prestressing force at transfer at a unit stress of 20 ksi and are placed within h/4 of the girder end, where h is the total girder depth. For a distance of 1.5d from the ends of the beams, reinforcement shall be placed to confine the prestressing steel in the bottom flange. The reinforcement shall be shaped to enclose the strands, shall be a #3 bar or greater and shall be spaced at less than or equal to 6". Note that the reinforcement shown on the Standard Details sheets satisfies these requirements.

Welded wire fabric may be used for the vertical reinforcement. It must be deformed wire with a minimum size of D18.

Per **LRFD [5.7.3.5]**, at the inside edge of the bearing area to the section of critical shear, the longitudinal reinforcement on the flexural tension side of the member shall satisfy:

$$A_s f_y + A_{ps} f_{ps} \geq \left(\frac{V_u}{\phi} - 0.5 V_s \right) \cot\theta$$

In the above equation, $\cot\theta$ is as defined in the V_c discussion above, and V_s is the shear reinforcement resistance at the section considered. Any lack of full reinforcement development shall be accounted for. Note that the reinforcement shown on the Standard Detail sheets satisfies these requirements.

19.3.3.17 Continuity Reinforcement

The design of non-prestressed reinforcement for negative moment at the support is based on the Strength I limit state requirements of **LRFD [5.6.3]**:

$$M_u = 1.25DC + 1.50DW + 1.75(LL + IM)$$

LRFD [5.5.4.2] allows a ϕ factor equal to 0.9 for tension-controlled reinforced concrete sections such as the bridge deck.

The continuity reinforcement consists of mild steel reinforcement in the deck in the negative moment region over the pier. Consider both the non-composite and the superimposed dead loads and live loads for the Strength I design of the continuity reinforcement in the deck.

Moment resistance is developed in the same manner as shown in 19.3.3.13.1 for positive moments, except that the bottom girder flange is in compression and the deck is in tension. The moment resistance is formed by the couple resulting from the compression force in the bottom flange and the tension force from the longitudinal deck steel. Consider A_s to consist of the longitudinal deck steel present in the deck slab effective flange width as determined in 19.3.3.8. The distance, d_p, is taken from the bottom of the girder flange to the center of the longitudinal deck steel.

WisDOT exception to AASHTO:

Composite sections formed by WisDOT standard prestressed I-girders shall be considered to be tension-controlled for the design of the continuity reinforcement. The ε_t check, as specified in **LRFD [5.6.2.1]**, is not required, and $\phi = 0.9$.

WisDOT policy item:

New bridge designs shall consider only the top mat of longitudinal deck steel when computing the continuity reinforcement capacity.

WisDOT policy item:

The continuity reinforcement shall be based on the greater of either the interior girder design or exterior girder and detailed as typical reinforcement for the entire width of the bridge deck. However, do not design the continuity steel based on the exterior girder design beneath a raised sidewalk. The continuity steel beneath a raised sidewalk should not be used for rating.

Based on the location of the neutral axis, the bottom flange compressive force may behave as either a rectangle or a T-section. On WisDOT standard prestressed I-girders, if the depth of the compression block, a, falls within the varying width of the bottom flange, the compression block acts as an idealized T-section. In this case, the width, b, shall be taken as the bottom flange width, and the width, b_w, shall be taken as the bottom flange width at the depth "a". During T-section behavior, the depth, h_f, shall be taken as the depth of the bottom flange of full width, b. See Figure 19.3-4 for details. Ensure that the deck steel is adequate to satisfy $M_r \geq M_u$.

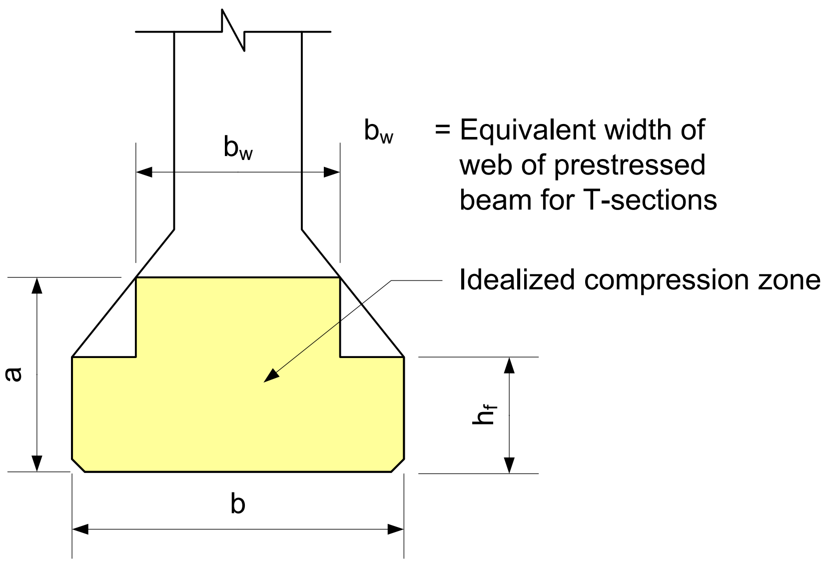

Figure 19.3-4
T-Section Compression Flange Behavior

The continuity reinforcement should also be checked to ensure that it meets the crack control provisions of **LRFD [5.6.7]**. This check shall be performed assuming severe exposure conditions. Only the superimposed loads shall be considered for the Service and Fatigue requirements.

The concrete between the abutting girder ends is usually of a much lesser strength than that of the girders. However, tests[1] have shown that, due to lateral confinement of the diaphragm concrete, the girder itself fails in ultimate negative compression rather than failure in the material between its ends. Therefore the ultimate compressive stress, f'_c, of the girder concrete is used in place of that of the diaphragm concrete.

This assumption has only a slight effect on the computed amount of reinforcement, but it has a significant effect on keeping the compression force within the bottom flange.

The continuity reinforcement shall conform to the Fatigue provisions of **LRFD [5.5.3]**.

The transverse spacing of the continuity reinforcement is usually taken as the whole or fractional spacing of the D bars as given in 17.5.3.2. Grade 60 bar steel is used for continuity reinforcement. Required development lengths for deformed bars are given in Chapter 9 – Materials.

WisDOT exception to AASHTO:

The continuity reinforcement is not required to be anchored in regions of the slab that are in compression at the strength limit state as stated in **LRFD [5.12.3.3]**. The following locations shall be used as the cut off points for the continuity reinforcement:

1. When ½ the bars satisfy the Strength I moment envelope (considering both the non-composite and composite loads) as well as the Service and Fatigue moment envelopes (considering only the composite moments), terminate ½ of the bars. Extend these bars past this cutoff point a distance not less than the girder depth or 1/16 the clear span for embedment length requirements.

2. Terminate the remaining one-half of the bars an embedment length beyond the point of inflection. The inflection point shall be located by placing a 1 klf load on the composite structure. This cut-off point shall be at least 1/20 of the span length or 4' from point 1, whichever is greater.

Certain secondary features result when spans are made continuous. That is, positive moments develop over piers due to creep[5], shrinkage and the effects of live load and dynamic load allowance in remote spans. The latter only exists for bridges with three or more spans.

These positive moments are somewhat counteracted by negative moments resulting from differential shrinkage[4] between the cast-in-place deck and precast girders along with negative moments due to superimposed dead loads. However, recent field observations cited in **LRFD [C5.12.3.3.2]** suggest that these moments are less than predicted by analysis. Therefore, negative moments caused by differential shrinkage should be ignored in design.

WisDOT exception to AASHTO:

WisDOT requires the use of a negative moment connection only. The details for a positive moment connection per **LRFD [5.12.3.3]** are not compatible with the Standard Details and should not be provided.

19.3.3.18 Camber and Deflection

The prestress camber and dead load deflection are used to establish the vertical position of the deck forms with respect to the girder. The theory presented in the following sections apply to a narrow set of circumstances. The designer is responsible for ensuring that the theoretical camber accounts for the loads applied to the girder. For example, if the diaphragms of a prestressed I-girder are configured so there is one at each of the third points instead of one at midspan, the term in the equation for $\Delta_{nc(DL)}$ related to the diaphragms in 19.3.3.18.2 would need to be modified to account for two point loads applied at the third points instead of one point load applied at midspan.

Deflection effects due to individual loads may be calculated separately and superimposed, as shown in this section. The *PCI Design Handbook* provides design aids to assist the designer in the evaluation of camber and deflection, including cambers for prestress forces and loads, and beam design equations and diagrams.

Figure 19.3-5 illustrates a typical prestressed I-girder with a draped strand profile.

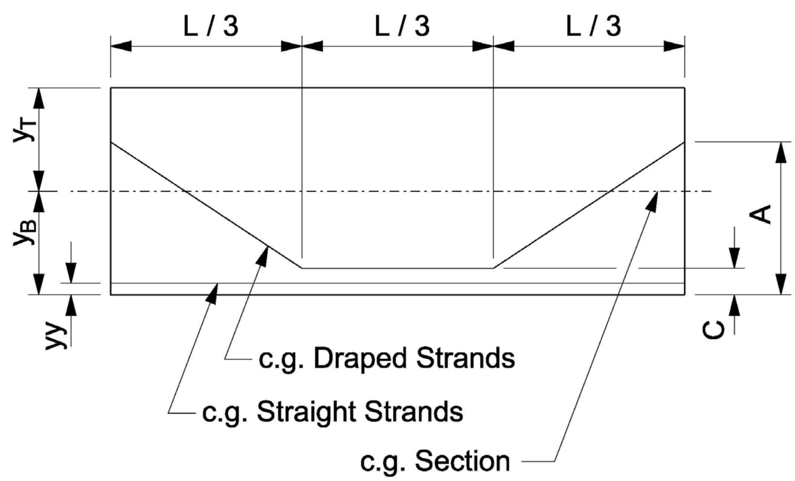

Figure 19.3-5
Typical Draped Strand Profile

19.3.3.18.1 Prestress Camber

The prestressing strands produce moments in the girder as a result of their eccentricity and draped pattern. These moments induce a camber in the girder. The values of the camber are calculated as follows:

Eccentric straight strands induce a constant moment of:

$$M_1 = \frac{1}{12}\left(P_i^s (y_B - yy)\right)$$

Where:

M_1	=	Moment due to initial prestress force in the straight strands minus the elastic shortening loss (k-ft)
P_i^s	=	Initial prestress force in the straight strands minus the elastic shortening loss (kips)
y_B	=	Distance from center of gravity of beam to bottom of beam (in)

yy	=	Distance from center of gravity of straight strands to bottom of beam (in)

This moment produces an upward deflection at midspan which is given by:

$$\Delta_s = \frac{M_1 L^2}{8E_i I_b} \quad \text{(with all units in inches and kips)}$$

For moments expressed in kip-feet and span lengths expressed in feet, this equation becomes the following:

$$\Delta_s = \frac{M_1 L^2}{8E_i I_b}\left(\frac{12}{1}\right)\left(\frac{12^2}{1}\right) = \frac{M_1 L^2}{8E_i I_b}\left(\frac{1728}{1}\right)$$

$$\Delta_s = \frac{216 M_1 L^2}{E_i I_b} \quad \text{(with units as shown below)}$$

Where:

Δ_s	=	Deflection due to force in the straight strands minus elastic shortening loss (in)
L	=	Span length between centerlines of bearing (ft)
E_i	=	Modulus of elasticity at the time of release (see 19.3.3.8) (ksi)
I_b	=	Moment of inertia of basic beam (in^4)

The draped strands induce the following moments at the ends and within the span:

$M_2 = \frac{1}{12}\left(P_i^D (A - C)\right)$, which produces upward deflection, and

$M_3 = \frac{1}{12}\left(P_i^D (A - y_B)\right)$, which produces downward deflection when A is greater than y_B

Where:

M_2, M_3	=	Components of moment due to initial prestress force in the draped strands minus the elastic shortening loss (k-ft)
P_i^D	=	Initial prestress force in the draped strands minus the elastic shortening loss (kips)
A	=	Distance from bottom of beam to center of gravity of draped strands at centerline of bearing (in)
C	=	Distance from bottom of beam to center of gravity of draped strands between hold-down points (in)

These moments produce a net upward deflection at midspan, which is given by:

$$\Delta_D = \frac{216L^2}{E_i I_b}\left(\frac{23}{27}M_2 - M_3\right)$$

Where:

Δ_D	=	Deflection due to force in the draped strands minus elastic shortening loss (in)

The combined upward deflection due to prestress is:

$$\Delta_{PS} = \Delta_s + \Delta_D = \frac{216L^2}{E_i I_b}\left(M_1 + \frac{23}{27}M_2 - M_3\right)$$

Where:

Δ_{PS}	=	Deflection due to straight and draped strands (in)

The downward deflection due to beam self-weight at release is:

$$\Delta_{o(DL)} = \frac{5W_b L^4}{384E_i I_b} \quad \text{(with all units in inches and kips)}$$

Using unit weights in kip per foot, span lengths in feet, E in ksi and I_b in inches^4, this equation becomes the following:

$$\Delta_s = \frac{5W_b L^4}{384E_i I_b}\left(\frac{1}{12}\right)\left(\frac{12^4}{1}\right) = \frac{5W_b L^4}{384E_i I_b}\left(\frac{20736}{12}\right)$$

$$\Delta_{o(DL)} = \frac{22.5W_b L^4}{E_i I_b} \quad \text{(with units as shown below)}$$

Where:

$\Delta_{o(DL)}$	=	Deflection due to beam self-weight at release (in)
W_b	=	Beam weight per unit length (k/ft)

Therefore, the anticipated prestress camber at release is given by:

$$\Delta_i = \Delta_{PS} - \Delta_{o(DL)}$$

Where:

Δ_i	=	Prestress camber at release (in)

Camber, however, continues to grow after the initial strand release. For determining substructure beam seats, average concrete haunch values (used for both DL and quantity calculations) and the required projection of the vertical reinforcement from the tops of the prestressed girders, **a camber multiplier of 1.4 shall be used**. This value is multiplied by the theoretical camber at release value.

19.3.3.18.2 Dead Load Deflection

The downward deflection of a prestressed I-girder due to the dead load of the deck and a midspan diaphragm is:

$$\Delta_{nc(DL)} = \frac{5W_{deck}L^4}{384EI_b} + \frac{P_{dia}L^3}{48EI_b} \quad \text{(with all units in inches and kips)}$$

Using span lengths in units of feet, unit weights in kips per foot, E in ksi, and I_b in inches^4, this equation becomes the following:

$$\Delta_s = \frac{5W_{deck}L^4}{384EI_b}\left(\frac{1}{12}\right)\left(\frac{12^4}{1}\right) + \frac{P_{dia}L^3}{48EI_b}\left(\frac{12^3}{1}\right) = \frac{5W_{deck}L^4}{384EI_b}\left(\frac{20736}{12}\right) + \frac{P_{dia}L^3}{48EI_b}\left(\frac{1728}{1}\right)$$

$$\Delta_{o(DL)} = \frac{22.5W_bL^4}{EI_b} + \frac{36P_{dia}L^3}{EI_b} \quad \text{(with units as shown below)}$$

Where:

$\Delta_{nc(DL)}$	=	Deflection due to non-composite dead load (deck and midspan diaphragm) (in)
W_{deck}	=	Deck weight per unit length (k/ft)
P_{dia}	=	Midspan diaphragm weight (kips)
E	=	Girder modulus of elasticity at final condition (see 19.3.3.8) (ksi)

A similar calculation is done for parapet and sidewalk loads on the composite section. Provisions for deflections due to future wearing surface shall not be included.

For girder structures with raised sidewalks, loads shall be distributed as specified in Chapter 17, and separate deflection calculations shall be performed for the interior and exterior girders.

19.3.3.18.3 Residual Camber

Residual camber is the camber that remains after the prestress camber has been reduced by the composite and non-composite dead load deflection. Residual camber is computed as follows:

$$RC = \Delta_i - \Delta_{nc(DL)} - \Delta_{c(DL)}$$

19.3.4 Prestressed I-Girder Deck Forming

Deck forming requires computing the relationship between the top of girder and bottom of deck necessary to achieve the desired vertical roadway alignment. Current practice for design is to use a minimum haunch of 2" at the edge of the girder flange. This haunch value is also used for calculating composite section properties. This will facilitate current deck forming practices which use 1/2" removable hangers and 3/4" plywood, and it will allow for variations in prestress camber. Also, future deck removal will be less likely to damage the top girder flanges. An average haunch height of 3 inches minimum can be used for determining haunch weight for preliminary design. It should be noted that the actual haunch values should be compared with the estimated values during final design. If there are significant differences in these values, the design should be revised. The actual average haunch height should be used to calculate the concrete quantity reported on the plans as well as the value reported on the prestressed girder details sheet. The actual haunch values at the girder ends shall be used for determining beam seat elevations.

For designs involving vertical curves, Figure 19.3-6 shows two different cases.

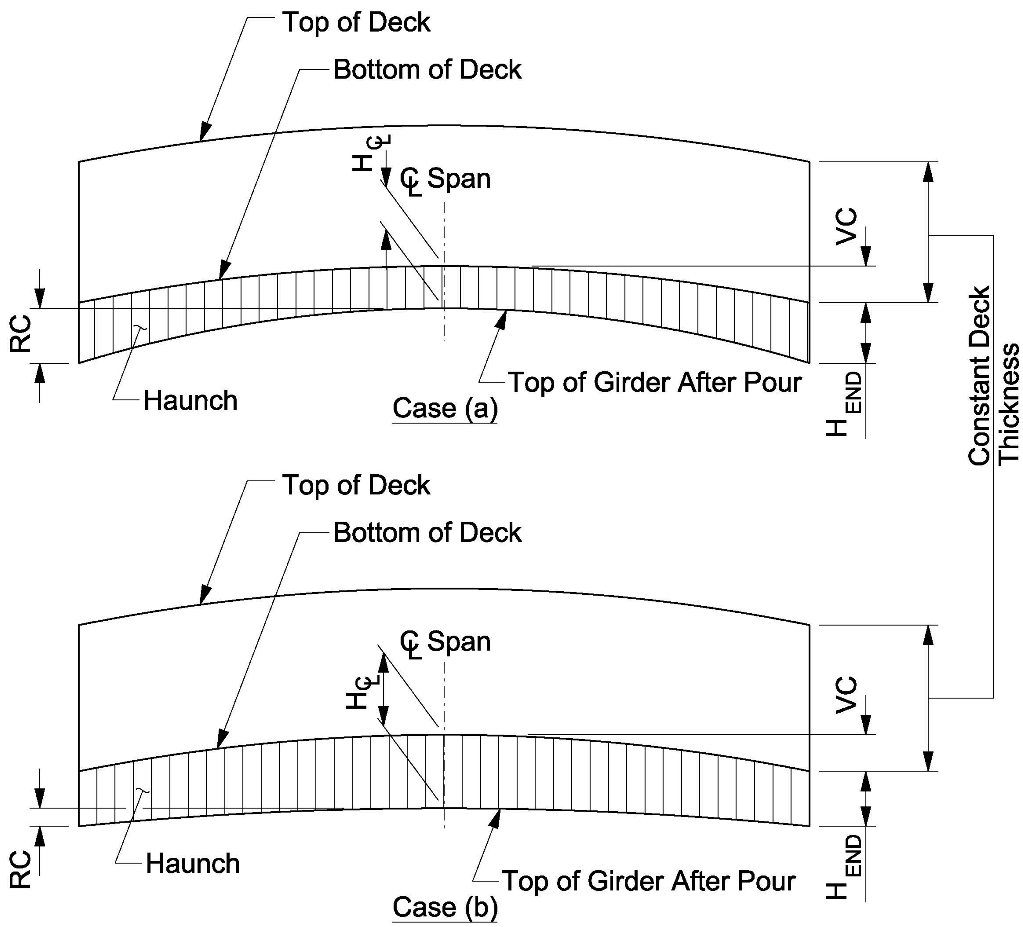

Figure 19.3-6
Relationship between Top of Girder and Bottom of Deck

In Case (a), VC is less than the computed residual camber, RC, and the minimum haunch occurs at midspan. In Case (b), VC is greater than RC and the minimum haunch occurs at the girder ends.

Deck forms are set to accommodate the difference between the bottom of the deck and the top of the girder under all dead loads placed at the time of construction, including the wet deck concrete and superimposed parapet and sidewalk loads. The deflection of superimposed future wearing surface and live loads are not included.

19.3.4.1 Equal-Span Continuous Structures

For equal-span continuous structures having all spans on the same vertical alignment, the deck forming is the same for each span. This is due to the constant change of slope of the vertical curve or tangent and the same RC per span.

The following equation is derived from Figure 19.3-6:

$$+H_{END} = RC - VC + (+H_{CL})$$

Where:

H_{END}	=	See Figure 19.3-6 (in)
RC	=	Residual camber, positive for upward (in)
VC	=	Difference in vertical curve, positive for crest vertical curves and negative for sag vertical curves (in)
H_{CL}	=	See Figure 19.3-6 (in)

19.3.4.2 Unequal Spans or Curve Combined With Tangent

For unequal spans or when some spans are on a vertical curve and others are on a tangent, a different approach is required. Generally the longer span or the one off the curve dictates the haunch required at the common support. Therefore, it is necessary to pivot the girder about its midspan in order to achieve an equal condition at the common support. This is done mathematically by adding together the equation for each end (abutment and pier), as follows:

$$(+H_{LT}) + (+H_{RT}) = 2[RC - VC + (+H_{CL})]$$

Where:

H_{LT}	=	H_{END} at left (in)
H_{RT}	=	H_{END} at right (in)

With the condition at one end known due to the adjacent span, the condition at the other end is computed.

19.3.5 Construction Joints

The transverse construction joints should be located in the deck midway between the cut-off points of the continuity reinforcement or at the 0.75 point of the span, whichever is closest to the pier. The construction joint should be located at least 1' from the cut-off points.

This criteria keeps stresses in the slab reinforcement due to slab dead load at a minimum and makes deflections from slab dead load closer to the theoretical value.

19.3.6 Strand Types

Low relaxation strands (0.5” and 0.6” in diameter) are currently used in prestressed I-girder and prestressed box girder designs and are shown on the plans. Strand patterns and initial prestressing forces are given on the plans, and deflection data is also shown.

19.3.7 Construction Dimensional Tolerances

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

19.3.8 Prestressed I-Girder Sections

WisDOT BOS employs two prestress I-girder section families. One I section family follows the AASHTO standard section, while the other 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 prestressed 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.

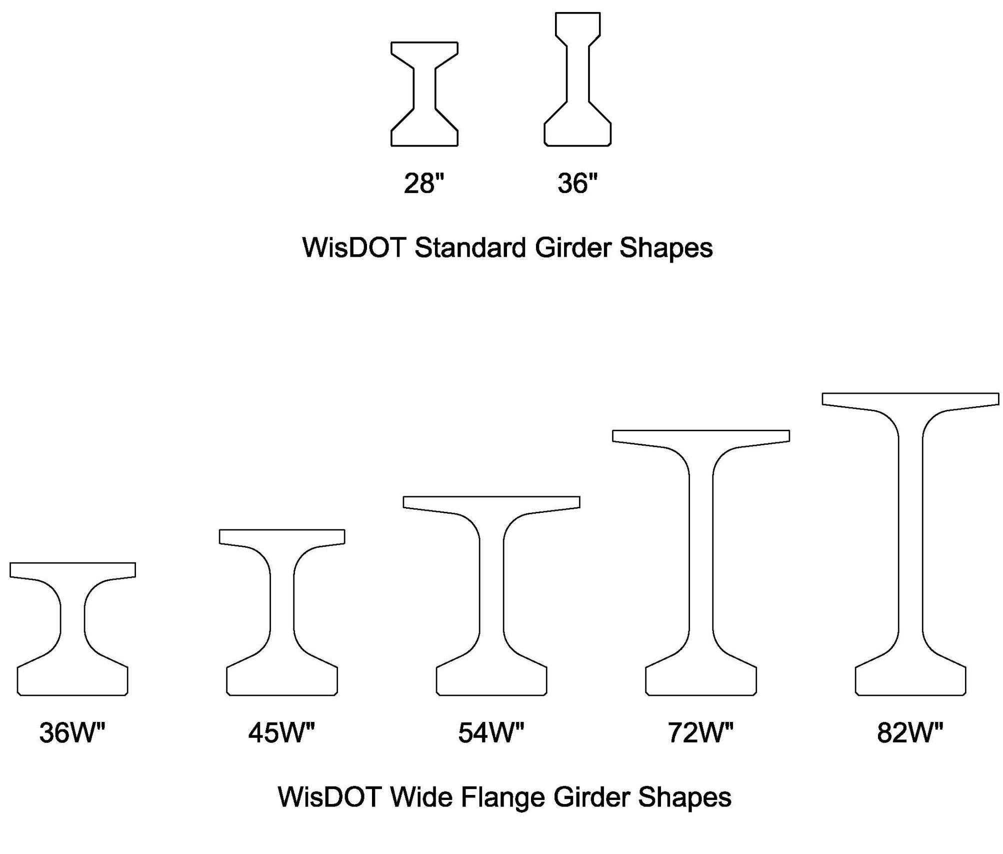

Figure 19.3-7
Prestressed I-Girder Family Details

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 I-girders.

Tables are based on:

- Interior prestressed 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

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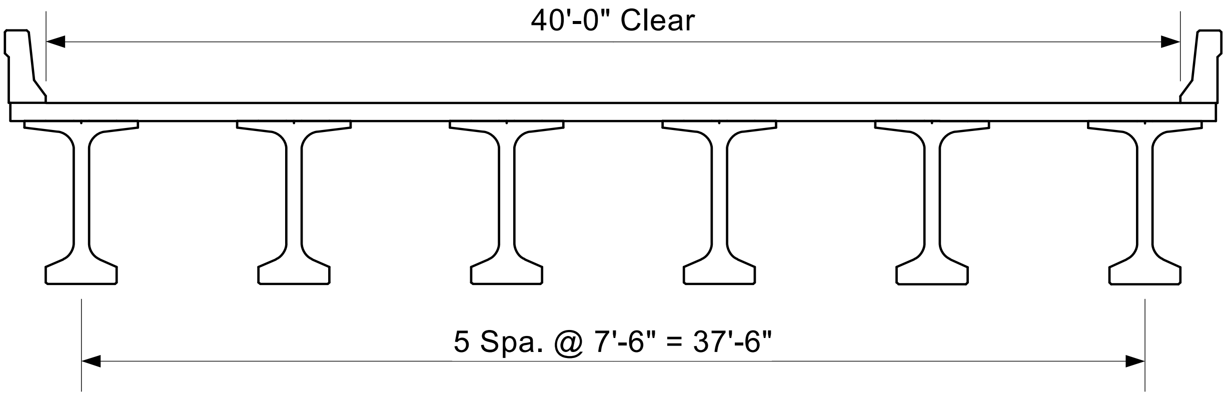
E19-1 Single Span Bridge, 72W" Prestressed Girders - LRFD

This example shows design calculations for a single span prestressed gider bridge. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. ***(Example is current through LRFD Eighth Edition - 2017)***

E19-1.1 Design Criteria



$L := 146$	center to center of bearing, ft
$L_g := 147$	total length of the girder (the girder extends 6 inches past the center of bearing at each abutment).
$w_b := 42.5$	out to out width of deck, ft
$w := 40$	clear width of deck, 2 lane road, 3 design lanes, ft
$f'_c := 8$	girder concrete strength, ksi
$f'_{ci} := 6.8$	girder initial concrete strength, ksi New limit for release strength.
$f'_{cd} := 4$	deck concrete strength, ksi
$f_{pu} := 270$	low relaxation strand, ksi
$d_b := 0.6$	strand diameter, inches
$A_{strand} := 0.217$	area of strand, in^2
$w_p := 0.387$	weight of Wisconsin Type LF parapet, klf
$t_s := 8$	slab thickness, in
$t_{se} := 7.5$	effective slab thickness, in
$skew := 20$	skew angle, degrees
$E_s := 28500$	ksi, Modulus of Elasticity of the Prestressing Strands
$w_c := 0.150$	kcf

E19-1.2 Modulus of Elasticity of Beam and Deck Material

Based on past experience, the modulus of elasticity for the precast and deck concrete are given in Chapter 19 as $E_{beam6} := 5500$ ksi and $E_{deck4} := 4125$ ksi for concrete strengths of 6 and 4 ksi respectively. The values of E for different concrete strengths are calculated as follows (ksi):

$$E_{beam8} := 5500 \cdot \frac{\sqrt{f'_c \cdot 1000}}{\sqrt{6000}} \qquad \boxed{E_{beam8} = 6351} \qquad E_B := E_{beam8}$$

$$n := \frac{E_B}{E_D} \qquad E_D := E_{deck4} \qquad \boxed{n = 1.540}$$

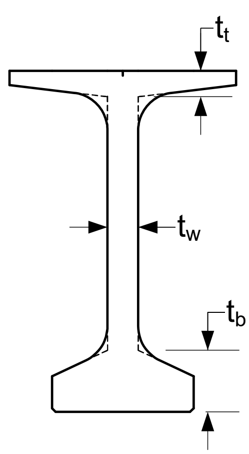
Note that this value of E_B is used for strength, composite section property, and long term deflection (deck and live load) calculations.

The value of the modulus of elasticity at the time of release is calculated in accordance with **LRFD [C5.4.2.4]**. This value of E_{ct} is used for loss and instantaneous deflection (due to prestress and dead load of the girder) calculations.

$$E_{beam6.8} := 33000 \cdot w_c^{1.5} \cdot \sqrt{f'_{ci}} \qquad \boxed{E_{beam6.8} = 4999} \qquad E_{ct} := E_{beam6.8}$$

E19-1.3 Section Properties

72W Girder Properties:



$w_{tf} := 48$	in		
$t_t := 5.5$	in		
$t_w := 6.5$	in		
$t_b := 13$	in		
$ht := 72$	in		
$b_w := 30$	width of bottom flange, in		
$A_g := 915$	in^2		
$r_{sq} := 717.5$	in^2	$y_b := -34.87$	in
$I_g := 656426$	in^4	$S_t := 17680$	in^3
$y_t := 37.13$	in	$S_b := -18825$	in^3

$$e_g := y_t + 2 + \frac{t_{se}}{2}$$ $\boxed{e_g = 42.88}$ in

Web Depth: $d_w := ht - t_t - t_b$ $\boxed{d_w = 53.50}$ in

$K_g := n \cdot \left(I_g + A_g \cdot e_g^{\,2}\right)$ **LRFD [Eq 4.6.2.2.1-1]** $\boxed{K_g = 3600866}$ in^4

E19-1.4 Girder Layout

Chapter 19 suggests that at a 146 foot span, the girder spacing should be approximately 7'-6" with 72W girders.

$S := 7.5$ ft

Assume a minimum overhang of 2.5 feet (2 ft flange + 6" overhang), $s_{oh} := 2.5$

$$n_{spa} := \frac{w_b - 2 \cdot s_{oh}}{S}$$ $\boxed{n_{spa} = 5.000}$

Use the next lowest integer: $n_{spa} := ceil(n_{spa})$ $\boxed{n_{spa} = 5}$

Number of girders: $ng := n_{spa} + 1$ $\boxed{ng = 6}$

Overhang Length: $s_{oh} := \frac{w_b - S \cdot n_{spa}}{2}$ $\boxed{s_{oh} = 2.50}$ ft

Recalculate the girder spacing based on a minimum overhang, $s_{oh} := 2.5$

$$S := \frac{w_b - 2 \cdot s_{oh}}{n_{spa}}$$ $\boxed{S = 7.50}$ ft

E19-1.5 Loads

$w_g := 0.953$ weight of 72W girders, klf

$w_d := 0.100$ weight of 8-inch deck slab (interior), ksf

$w_h := 0.125$ weight of 2.5-in haunch, klf

$w_{di} := 0.460$ weight of diaphragms on interior girder (assume 2), kips

$w_{dx} := 0.230$ weight of diaphragms on exterior girder, kips

$w_{ws} := 0.020$ future wearing surface, ksf

$w_p = 0.387$ weight of parapet, klf

E24-2.11 Draw Schematic of Final Bolted Field Splice Design

Figure E24-2.11-1 shows the final bolted field splice as determined in this design example.

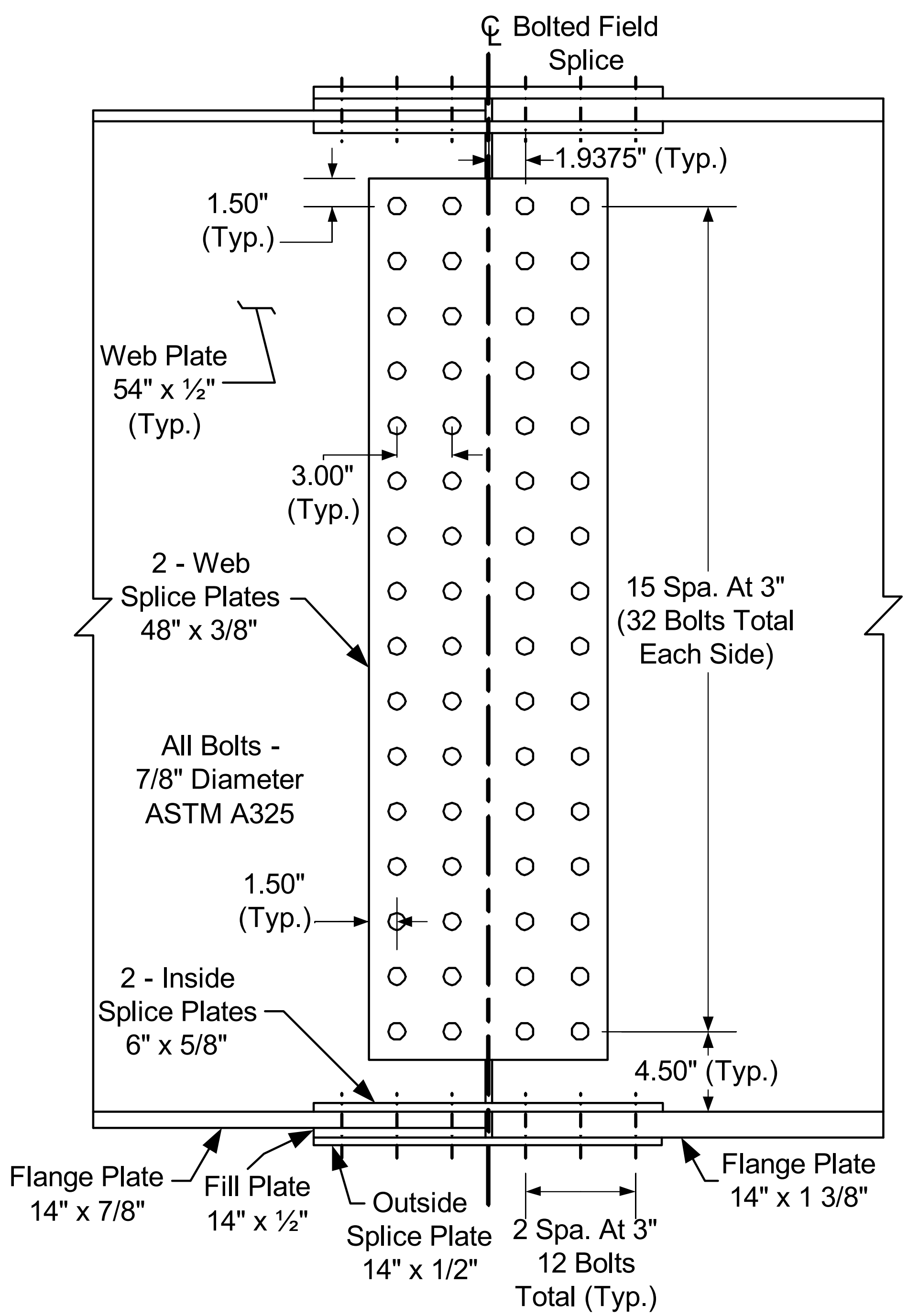


Figure E24-2.11-1
Final Bolted Field Splice Design

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39.1 General

39.1.1 Introduction

Signing is an integral part of the highway plan and as such is developed with the roadway and bridge design. Sign support structures are divided into two categories: Roadside signs, and Overhead Sign Structures (OSS). Roadside signs are designed and specified by the roadway engineer. OSS are designed by a Department (in-house or consultant) structural engineer or by a contractor, depending on the type of OSS.

Generally, an OSS is comprised of three components: the sign(s), the structure, and the foundation. Signage details are provided in the WisDOT Sign Plate Manual referenced below. This chapter of the WisDOT Bridge Manual (BM) governs the design of the structure and the foundation for OSS.

Regional traffic engineers determine the type of overhead sign structure that meets the signage needs for a particular project. Selection guidance and information is provided in the Facilities Development Manual (FDM) 11-55-20. That selection is communicated to the Bureau of Structures through the SSR submittal process.

The responsibility for developing contract plans depends on the type of sign structure selected and may be the role of Bureau of Structure staff, Regional staff, or engineering consultants.

39.1.2 Sign Structure Types and Definitions

Roadside Sign: Refers to roadside signs supported on ground mounted posts adjacent to roadways. Ground mounted sign support posts are not considered "structures" and as such, are not assigned a structure number. See WisDOT Sign Plate Manual for details.

https://wisconsindot.gov/Pages/doing-bus/local-gov/traffic-ops/manuals-and-standards/signplate/signplate.aspx

Overhead Sign Structure (OSS): Refers to structural supports for mounting signs over a roadway. OSS are assigned a structure number and inventoried in WisDOT's Highway Structures Information (HSI) system. These structures are included on the section 8 structure sheets of a contract plan set.

In prior editions of the Bridge Manual there were two categories of overhead sign structures - "Sign Bridges" and "Overhead Sign Supports (OHSS)". Sign bridges were Department designed, and OHSS were contractor designed. While the roles of design remain the same, this edition shifts away from that terminology, instead focusing on terminology that best describes the geometric characteristics of the sign structure.

Table 39.1-1 summarize OSS types used by WisDOT:

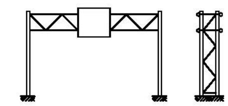
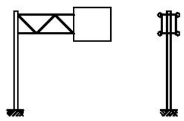
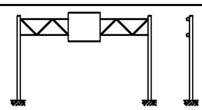
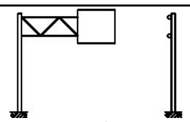
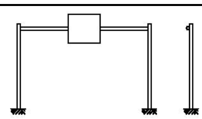
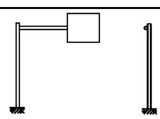
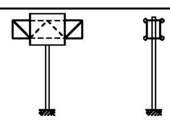
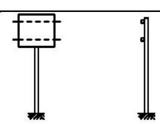
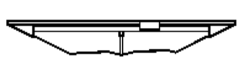
Overhead Sign Structure Type	Description	Standard Structure Design	Standard Foundation Design
 Full Span 4-Chord Truss	A 4-chord space truss with dual, trussed vertical support posts at each end. Used to support large Type I static highway sign panels and Dynamic Message Signs (DMS). Typically used over multi-lane state highways and interstate routes.	Yes	Yes
 Cantilever 4-Chord Truss	A 4-Chord space truss with a single vertical support post. Used to support large Type I static highway sign panels and DMS. Commonly used to span over the outside lanes of multi-lane state highways and interstate routes to delineate exit lanes and ramps.	Yes	Yes
 Full Span 2-Chord Truss	A 2-chord planar truss with single vertical support posts at each end. Used to support Type II and smaller Type I static signs and DMS over roadways and state highways.	No	Yes
 Cantilever 2-Chord Truss	A 2-chord planar truss with a single vertical support post. Used to support Type II and smaller Type I static signs and DMS over roadways and state highways.	No	Yes
 Full Span Monotube	Similar to a Full Span 2-Chord Truss but with only a single horizontal sign support member. Used to support small Type II static signs.	No	Yes
 Cantilever Monotube	Similar to a Cantilever 2-Chord Truss but with only a single horizontal support member. Used to support small Type II static signs.	No	Yes
 Butterfly Truss	A 4-Chord space truss with a centrally located single vertical support post used to support DMS. Typically used in the medians of multi-lane interstate routes.	No	No
 Butterfly	Similar to a Butterfly Truss but with multiple monotube horizontal sign support members.	No	No
 Bridge Mounted Sign Support	Sign support brackets to mount signs to the sides of grade separation highway bridges over the underpass roadway. These are typically used in special circumstances where other OSS types cannot be used.	No	NA
Note: Tri-chord and dual non-trussed monotubes are not permitted (except for Butterfly type)			

Table 39.1-1
WisDOT Overhead Sign Structure Types

39.1.3 Additional Terms

Type I Sign: Larger signs on an extruded aluminum base material, typically mounted on steel I-beams. Large guide and message signs with green backgrounds on interstate routes are Type I signs.

Type II Sign: Signs consisting of direct applied message on either plywood or sheet aluminum base material, typically mounted on wood or steel posts.

Dynamic Message Sign (DMS): An electronic traffic sign, often used in urban settings to inform drivers of unique and variable information. These signs are generally smaller in wind loaded area than Type I signs, but are heavier and load the truss eccentrically.

OSS Standard Designs: A group of pre-designed sign structures. The standard design includes both the structure and its foundation. The limitations for use is provided in section 39.1.5 and 39.1.6. See for further information on OSS Standard Designs.

OSS Non-Standard Design: Refers to sign structures that fall outside the OSS Standard Design parameters. It also applies to sign structure types not covered by standard design. These sign structures require a structural engineer provide a unique individual design of the structure and/or its foundation. See 39.4.5 for further information on OSS Non-standard Designs.

OSS Contractor Designed: Refer to sign structures that are designed and detailed by the contractor as part of the construction contract. The limitations for use is provided in section 39.1.5 and 39.1.6. The contractor does not design the foundation. For this, pre-designed foundations are available for use with these types of sign structures. See 39.4.6 for further information on OSS Contractor Designed.

OSS Standard Design Drawings: Refers to a library of WisDOT developed detail drawings for the *OSS Standard Designs* and the foundations for *OSS Contractor Designed*, otherwise indicated by a “yes” in Table 39.1-1. These standard design drawings are inserted into the contract plans with no additional design or detailing effort required.

39.1.4 OSS Selection Criteria

Chapter 11-55-20 of the Facilities Development Manual (FDM) provides selection guidance for determining sign structure type. The selection guidance was developed based on the design limitations of Table 39.1-1 and Table 39.1-2 and the information provided in the OSS Standard Design Drawings.

39.1.5 Cantilever OSS Selection Criteria

Cantilever OSS Type	Design	Cantilever Length [1]	Vertical Support Height [1]	Static Sign Total Area & Max. Dimensions [2]		DMS Total Area & Weight [1]
Monotube	Contractor Designed	40'-0" Max.	25'-0" Max. Column Base Plate to CL of Monotube Arm	Sign Area ≤ 75 SF Max. Sign Height ≤ 5'-0"		Not Used
2-Chord Truss	Contractor Designed	40'-0" Max. (static) / 20'-0" Max. (DMS)	27'-0" Max. Column Base Plate to CL of Top Chord	Sign Area ≤ 150 SF Max. Sign Height ≤ 10'-0"		13'-9"W x 8'-0"H Max. 750 Lbs. Max
4-Chord Truss	Standard Design	20'-0" Min. 30'- 0" Max.[2]	30'-0" Max. Column Base Plate to CL of Top Chord	Sign Area ≤ 264 SF Max. Sign Height ≤ 15'-0"	OR	9'-0"W x 6'-0"H 2,500 Lbs. Max.
4-Chord Truss	Standard Design	>30'-0" 38'-0" Max.[2]	30'-0" Max. Column Base Plate to CL of Top Chord	Sign Area ≤ 240 SF Max. Sign Height ≤ 15'-0"		9'-0"W x 6'-0"H 2,500 Lbs. Max.
4-Chord Truss	Non-Standard Design	>38'-0"	Column Height Exceeds Limit for Standard Design	Sign Area or Max. Sign Height Exceeds Limits For Standard Design		DMS Dimensions or Weight Exceeds Limits For Standard Design

Table 39.1-2
Cantilever OSS Selection Criteria

Note 1: The limiting parameters of length, height and sign area are depicted in 39.6 for Contractor Designed OSS and on the Standard Design Drawings for the 4-Chord OSS.

Note 2: Static Type I sign panels may extend 1'-0" beyond end of Cantilever 4-Chord Truss.

39.1.6 Full Span OSS Selection Criteria

Full Span OSS Type	Design	Span Length [1]	Vertical Support Height [1]	Static Sign Total Area & Max. Dimensions		DMS Max. Dimensions & Max. Weight [1]
Monotube	Contractor Designed	40'-0" Min. 75'-0" Max.	25'-0" Max. Column Base Plate to CL of Monotube Arm	Sign Area ≤ 150 SF Max. Sign Height ≤ 5'-0"		Not Used
2-Chord Truss	Contractor Designed	40'-0" Min. 100'-0" Max. (static) / 70'-0" Max. (DMS)	27'-0" Max. Column Base Plate to CL of Top Chord	150 SF < Sign Area ≤ 300 SF Max. Sign Height ≤ 10'-0"	OR	10'-6"W x 6'-0"H Max. 850 Lbs. Max
4-Chord Truss	Standard Design	40'-0" Min. 130'-0" Max.	30'-0" Max. Column Base Plate to CL of Top Chord	300 SF < Sign Area ≤ Note 2 Max. Sign Height ≤ 12'-0"		26'-0"W x 9'-0"H 4,500 Lbs. Max.
4-Chord Truss	Non-Standard Design	>130'-0"	Column Height Exceeds Limit for Standard Design	Sign Area or Height Exceeds Limits For Standard Design		DMS Dimensions or Weight Exceeds Limits For Standard Design

Table 39.1-3
Full Span OSS Selection Criteria

Note 1: The limiting parameters of length, height and sign area are depicted in 39.6 for Contractor Designed OSS and on the Standard Design Drawings for the 4-Chord OSS.

Note 2: Maximum sign area for full span 4-chord standard design = 12' x (90% * Span Length).

39.1.7 Butterfly and Butterfly Truss OSS

OSS Type	Design	Static Sign Total Area & Max. Dimensions[2]		DMS Total Area & Weight
Butterfly	Non-Standard Design	Sign Area ≤ 200 Sq. Ft. Sign Height ≤ 10'-0"	OR	N.A.
Butterfly Truss [1]	Non-Standard Design	Sign area > 200 sq. ft. Sign Height > 10'-0"		See 4-Chord full span requirements. Limit 2 per structure.

Table 39.1-4
Butterfly and Butterfly Truss OSS Selection Criteria

Note 1: Butterfly Trusses should use the WisDOT 4-chord cantilever truss dimensions (3'-9"W x 5'-0"H). Details similar to the 4-chord cantilever should be used in the design of these structures

Note 2: The above sign areas are for one side only. Butterfly and Butterfly Truss structures can have double the total sign area listed with back to back signs mounted on each side of the structure.

39.1.8 Design Process

The design process for sign structures generally follows the process for bridge structures as detailed in chapter 6. There are some notable exceptions. First, the design of sign structures are usually initiated later in the overall process because they are dependent on a fairly established roadway plan. Second, a certain subset of sign structure types are permitted to be designed and detailed by a contractor, with other types requiring a department structural engineer (in-house or consultant) providing the design and detailing.

As outlined in 11-55-20.3 of the FDM, the Region initiates the sign structure design process by submitting to BOS an SSR. For *Contractor Designed* or *Standard Design* OSS types, as defined in 39.1.3, the Region or their consultant prepare final contract plans and submits via the structure e-submit process at least two months prior to PS&E. BOS must be notified if there are changes to the sign structure type after the SSR is submitted.

Region or consultant staff assemble final contract plans using the lead sheet templates and the *OSS Standard Design Drawings*, available on the BOS website under the Chapter 39 Bridge Standards - *LRFD Standardized Plans.* See 39.4.4 and 39.4.6 for more information on preparing standardized plans.

Involvement of a Department structural engineer in the design and detailing of individual sign structures is generally limited to *Non-standard* design types. If a Non-standard design is warranted, for reasons detailed in 39.4.5, then the design process follows the normal flow as defined in Chapter 6, requiring either BOS design staff or an engineering consultant provide a unique design and the final contract plans. Non-standard designs should make use of the OSS Standard Design Drawings where appropriate.

39.2 Materials

WisDOT uses the following for round, hollow structural sections (HSS) for truss chord members, vertical support members and horizontal monotube members.

Member Type	Material Requirements	
HSS Chords, Vertical Supports, & Horizontal Monotubes	Wall Thickness ≤ ½”	ASTM A500, Grade C (F_y = 46 ksi)
	Wall Thickness > ½” and Pipe Diameter ≤ 20”	ASTM A1085 (round HSS) Or API 5L Grade 46 PSL-2 (round pipe)
	Pipe Diameter > 20” (Any Wall Thickness)	API 5L Grade 46 PSL-2 (round pipe)
Plates, Bars, and Structural Angles	ASTM A709, Grade 36	
Round or Multi-Sided Tapered Poles	ASTM A595, Grade 55 Or ASTM A575, Grade 55	

All plates, bars and structural angles shall be ASTM A709 Grade 36.

Round or multi-sided tapered poles shall be ASTM A595 Grade 55 or A575 Grade 55.

Galvanized ASTM F3125 A325 bolts with DTI washers are to be used in all primary structural connections, including those that are fully tensioned. A449 bolts are not allowed in fully tensioned connections and are only allowed in full span chord to column saddle or full span post to chord clamp connections. More details can be found in the OSS Standard Design Drawings and Standard Specifications Section 532.

<u>WisDOT policy item:</u>

Installation of flat washers in between faying surfaces of mast arm connection plates is not allowed.

When selecting members sizes for individually designed OSS, it is important to select members that are regularly produced and domestically available. Specifying members that are infrequently produced may result in higher bid prices, longer fabrication lead time, and/or member substitution requests that may delay the fabrication and production process. A general rule of thumb is to select HSS round tube members that match standard (Schedule 40) outside pipe diameters and thickness. The Steel Tube Institute provides current information on their website regarding domestic availability of HSS sections at:

https://steeltubeinstitute.org/hss/availability-tool/.

Designers can also consult the Bureau of Structures.

39.3 Specifications

39.3.1 LRFD Design

WisDOT has transitioned the design of all roadside standard Type 1 breakaway sign supports and foundations to be in accordance with the *AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Signals, 1st Edition (2015) (LRFDLTS-1)* with current Interim revisions.

WisDOT is currently transitioning the design of OSS to be in accordance with the AASHTO LRFDLTS-1 with current Interim revisions. Use of the AASHTO LRFDLTS-1 specification for OSS is currently optional and will be required beginning with the December 2020 letting.

39.3.2 Other Specifications and Manuals

The following manuals and specifications provide further guidance and requirements for the design and construction of OSS:

- Wisconsin Department of Transportation "*Bridge Manual*" (BM)
- Wisconsin Department of Transportation "*Geotechnical Manual*"
- Wisconsin Department of Transportation *"Facilities Development Manual"* (FDM)
- State of Wisconsin *"Standard Specifications for Highway and Structure Construction"*
- State of Wisconsin "*Construction and Materials Manual*" (CMM)
- AASHTO "*LRFD Bridge Design Specifications"* (Current Edition and Interim Specifications)
- American Society for Testing and Materials Standards (ASTM)
- American National Standards Institute / American Petroleum Institute 5L Specification for Line Pipe. (ANSI / API 5L)
- AWS D1.1 Structural Welding Code (Steel)
- AWS D1.2 Structural Welding Code (Aluminum)

39.4 Design Considerations

39.4.1 Roadside Signs

Supports for roadside signs are of two types, depending upon the size and type of the sign to be supported. For small signs, the column supports are treated timber embedded in the ground. For larger Type 1 signs and DMS, the columns are galvanized steel supported on drilled shafts. Standard design and support estimates are given in the A3 Series of the "Sign Plate Manual."

Roadside sign supports are located behind existing or planned guardrail as far as practical from the roadway and out of the likely path of an errant vehicle. If roadside signs are located within the 30-foot corridor and not protected, break-away sign supports are detailed. Roadside sign supports for DMS, which includes dynamic message signs and variable message signs, are to be protected by concrete barrier or guardrail.

Currently, all steel column supports for roadside Type 1 signs, and DMS are designed to break-away upon impact.

The Wisconsin DOT Bureau of Traffic Operations has standard designs and details available for Type 1 Roadside Sign supports and foundations. The standard steel post design tables provide maximum sign mounting heights. If a sign configuration is required that does not fall within the limits of the standard designs, the sign support must be designed by a structural engineer. The design must be in compliance with the applicable specifications listed in 39.3. The Type 1 roadside sign standard foundation designs are based on the assumptions of cohesionless soils with the following properties:

- Soil Unit Weight = 115 pounds per cubic foot
- Angle of Internal Friction = 24 degrees
- Soil Modulus Parameter = 25 pounds per cubic inch

Wisconsin has standard design and details available for DMS roadside sign supports. If weaker subsurface conditions are known or suspected, a subsurface soil investigation per 39.5 would be implemented to gather necessary design information.

39.4.2 Overhead Sign Structures (OSS)

39.4.2.1 General

OSS types and names used by WisDOT are summarized in Table 39.1-1.

The connections of web members to chords are designed for bolted or shop welded connections to allow the contractor the option to either galvanize individual members or complete truss sections after fabrication. Steel base plates are used for anchor rod support attachment.

Aluminum sign structures are currently not being designed for new structures. Rehabilitation and repair type work may require use of aluminum members and shall be allowed in these limited instances. The following guidelines apply to aluminum structures in the event of repair and rehabilitation type work.

Aluminum sign structure trusses are designed and fabricated from tubular shapes shop welded together in sections. The minimum thickness of truss chords is 1/4-inch and the minimum outside diameter is 4 inches. The recommended minimum ratios of “d/D” between the outside diameters “d” of the web members and “D” of chord members is 0.4. A cast aluminum base plate is required to connect the aluminum columns to the anchor rods. AASHTO Specifications require damping or energy absorbing devices on aluminum overhead sign support structures to prevent vibrations from causing fatigue failures. Damping devices are required before and after the sign panels are erected on all aluminum sign bridges. Stock-bridge type dampers are recommended.

39.4.2.2 Vehicular Protection

Vertical supports for OSS Standard Designs are not designed for vehicular impact loads and must meet clear zone or barrier protection requirements in the FDM. Generally, all overhead sign structure vertical supports are located at the edge of shoulder adjacent to the traveled roadway and placed behind roadside concrete barriers or barrier type guardrail. See the FDM 11-55-20.6 for information on shielding requirements. Sign supports protected by roadside barriers or guardrail with adequate barrier deflection clearance between the backside of the barrier and the sign support are not required to be designed for Extreme Limit State vehicular collision loads.

When protection is not feasible, the vertical supports shall be designed with applicable Extreme Event collision loads in accordance to 13.4.10. This typically requires the use of a special, individually designed reinforced column and foundation to resist the large vehicular impact loads. In this situation the sign structure would be a non-standard design and BOS or an engineering consultant would need to provide the design.

39.4.2.3 Vertical Clearance

As provided in the FDM 11-35-1 Attachment 1.8, a minimum vertical clearance of 18’-3” is required for most routes. For sign structures over a designated High Clearance Route, 20’-3” above the roadway is required. See FDM 11-35-1 Attachment 1.9 for clearances relating to existing sign structures.

39.4.2.4 Lighting and DMS Inspection Catwalks

Lighting is no longer required on sign structures. Catwalks are only on 4-chord cantilever and full span OSS with DMS. When catwalks are provided for OSS supporting a DMS, additional vertical height must be provided to meet the vertical clearance requirements in 39.4.2.3 to the bottom of the catwalk brackets. Catwalk grating and toe plates shall be galvanized steel.

Along with inspection catwalks, all DMS OSS require hand holes, rodent screens and electrical conduits through the foundation to one of the vertical support posts to route electric power to the DMS. Standard Details are provided on the BOS website.

39.4.2.5 Signs Mounted on the Side of Grade Separation Bridges

When no practical alternatives exist, signs may be mounted on the side of grade separation bridges. This application requires individually designed structural mounting brackets to attach the sign to the side of the grade separation bridge. Wisconsin allows sign attachments orientated up to a maximum of a 20-degree skew to the roadway below. Any grade separation bridge structure with greater skew requires the mounting brackets to attach signs so they are orientated perpendicular to the roadway below.

Where possible, the depth of bridge mounted signs should be limited so the top of the sign does not extend above the top of the bridge parapets or railing. Signs are not permitted to extend below the bottom of the bridge girders. The minimum vertical clearance is set slightly above the clearance line of the overpass structure for signs attached directly to the side of a bridge.

Signs attached to the side of bridge superstructures and retaining walls are difficult to inspect and maintain due to lack of access. Attachment accessories are susceptible to deterioration from de-icing chemicals, debris collection and moisture. Therefore, the following guidance should be considered when detailing structure mounted signs and related connections:

1. Limit the sign depth to a dimension equal to the bridge superstructure depth (including parapet) minus 3 inches.

2. Provide at least two support connections per bracket.

3. Utilize cast-in-place anchor assemblies to attach sign supports onto new bridges and retaining walls.

4. Galvanized or stainless-steel adhesive concrete masonry anchor may be used to attach new signs to the side of an existing grade separation bridge or retaining wall orientated for shear load application only. Overhead anchor installation (direct pullout loading on anchor) is not allowed. Reference 40.16 for applicable concrete masonry anchor requirements.

39.4.2.6 Sign Structures Mounted on Bridge Pedestals

This refers to sign structures mounted across the top of roadways carried by a bridge structure. Sign structures can be mounted directly to the top of pier caps. This requires the pier cap to be extended beyond the limits of the superstructure width. Sign structures mounted to pier caps are not affected by superstructure deflections. Wisconsin allows sign attachments oriented up to a maximum of a 20-degree skew to the roadway below. Any grade separation bridge structure with a greater skew requires the mounting brackets to attach signs so they are oriented perpendicular to the roadway below.

Span live load deflections of the vehicular bridge superstructure affect sign structures mounted on to bridge superstructure concrete barrier pedestals. The magnitude of sign structure deflections and duration of sign structure vibrations is dependent on the stiffness of the girder and deck superstructure, the location of the sign structure on the bridge, and the ability of the sign structure to dampen those vibrations out; among others. These vibrations are not easily accounted for in design and are quite variable in nature. For these reasons, the practice of locating sign structures on highway bridges should be avoided whenever possible.

The following general guidance is given for those instances where locating a sign structure on a bridge structure is unavoidable. This may occur due to the length of the bridge, or a safety need to guide the traveling public to upcoming ramp exits or into specific lanes on the bridge.

1. Locate the sign structure pedestals at pier locations.

2. Build the sign structure base off the top of the pier cap.

3. Provide adequate set back of the tower support of the sign structure behind the face of the parapet to avoid snagging of vehicles making contact with the parapet. See FDM 11-45-2.3.6.2.3 for information on required set back distances.

4. Use single pole sign supports (equal balanced butterfly's) in lieu of cantilevered (with an arm on only one side of the vertical support) sign supports.

5. Consider the use of a Stockbridge type damper in the horizontal truss of these structures.

6. Do not straddle the pier leaving one support on the pier and one support off the pier in the case of skewed substructure units for full span sign bridges

39.4.3 LRFD Requirements and WisDOT Guidance for OSS Design

39.4.3.1 Loads, Load Combinations, and Limit States

All OSS are to be designed per the AASHTO LRFDLTS-1. The following LRFD specification requirements are highlighted:

Design Wind Speed Recurrence Interval:

- Full Span 4-Chord Truss Sign Structures are designed for a basic wind speed recurrence interval of 1,700 years as defined in the AASHTO LRFDLTS-1 Specifications.

- All other OSS shall be designed for a basic wind speed recurrence interval of 700 years as defined in the AASHTO LRFDLTS-1 Specifications.

Wind load and wind load combinations shall be applied and investigated per AASHTO LRFDLTS-1. In general, horizontal wind pressure is applied normal to the center of gravity of exposed horizontal members and sign panels. For the design of vertical supports, three wind load cases are investigated and applied to the entire structure to determine the controlling wind load effect on the vertical supports:

Wind Load Case	Description	Normal Wind Component	Transverse Wind Component
1	Full Wind Normal to the Plane of the Structure	100%	0%
2	Full Wind Transverse to the Plane of the Structure	0%	100%
3	75% Full Wind in Both Directions Simultaneously	75%	75%

Figure 39.4-1
AASHTO LRFDLTS-1 Vertical Support Load Cases

Design sign area assumed for standard designs accommodates 12-foot high sign panels over 90% of the span length for full span 4-chord truss OSS. In the case of a proposed non-standard OSS with a required span length of 130 feet or less, the non-standard OSS should be individually designed for the actual anticipated sign panel area or DMS and mounting locations. In the case of a proposed non-standard OSS with required span length greater than 130 feet, the Bureau of Structures should be consulted to confirm the design sign area to be used for the design of the non-standard OSS.

Applied design wind pressure is determined for individual truss members and sign panels per the AASHTO LRFDLTS-1 specification Section 3.8. WisDOT design practice is to assume members located directly behind sign or DMS panels are shielded from wind exposure and are not loaded with wind pressure. No wind shielding is assumed provided to members that directly align with each other in plan or elevation views, but are several feet apart. This means no shielding effect is assumed for members in the front and back truss planes of a 4-chord truss even if the members are perfectly aligned. For example, viewing a 4-chord truss in elevation view, members in the front truss plane, located directly behind a sign panel would assumed to be shielded from wind pressure by the sign panel, but members in the back-truss plane would assumed to be loaded with wind pressure, despite also being behind the sign panel or aligned with other members in the front truss plane.

Strength 1 load combinations in AASHTO LRFDLTS-1 include only dead load and live load. A 500-pound live load distributed over 2’-0” transversely to the member, only applies to catwalks and catwalk support brackets when catwalks are included for OSS with DMS. The Strength 1 load combination may control the design of the catwalk and catwalk support brackets, but does not control the design of the truss superstructure the catwalk brackets are attached to. For OSS carrying static Type 1 signs, the Strength I load combination includes only dead load and does not control.

Load combinations that include wind generally control the design of sign structures. A change in the AASHTO LRFDLTS-1 specification is that load combinations that include wind are considered Extreme Event load cases.

AASHTO LRFDLTS-1 specifications do not define an ice loading and leave it to the discretion of individual owners to consider and specify an ice loading if warranted in their climate. WisDOT policy is to maintain consideration of an ice load and include in the Extreme Event I load combination.

Load Combinations are as follows:

Strength I:	1.25 DL + 1.6 LL
Extreme Event I (Load Case 1):	1.1 DL + 1.0 ICE + W (Max. DL and ICE effects)
Extreme Event I (Load Case 2):	0.9 DL + W (Min. DL and no ICE effects)

Ice build-up is modeled as a 3 psf load applied to the exposed surface area (circumference) of truss members. It is not necessary to increase the wind pressure load on truss members due to increased member exposure area caused by ice build-up. Ice load is applied to only the front face of sign or DMS panels.

For vertical column support members, W in the above Extreme Event load cases is the controlling wind load case as specified in Figure 39.4-1.

39.4.3.2 Serviceability

Serviceability checks should conform to 10.4 and 10.5 of AASHTO LRFDLTS-1. However, the vertical deflection limit L/150 shall include ICE load, applied per 39.4.3.1.

39.4.3.3 Fatigue

AASHTO LRFDLTS-1 specifies three fatigue loads to check against member and connection fatigue stress range limits as follows:

Galloping – AASHTO LRFDLTS 11.7.1.1:	Applies to all cantilever OSS, except cantilever 4-chord truss
Natural Wind Gust – AASHTO LRFDLTS 11.7.1.2:	Applies to all OSS.
Truck Induced Gust – AASHTO LRFDLTS 11.7.1.3:	Applies to all OSS.

Truck induced gust pressure is applied in the upward direction and reduces with increasing height. Truck induced gust pressure applied to truss members in the top horizontal truss plane, will be less than truck induced gust pressures applied to truss members in the bottom horizontal truss plane. Since truck induced gust pressure is acting upward, Type 1 static signs receive and transmit only minimal gust pressure due to their narrow profile when viewed in plan. DMS however, have considerable width and “wind exposure area” when viewed in plan. Truck induced gust pressure can impart a significant upward pressure on DMS that also creates a torque on the truss superstructure due to the offset between the center of gravity of the DMS and the truss superstructure.

39.4.3.4 Connection Design

WisDOT policy item:

Bureau of Structures policy is to design welded and bolted connections per the applicable provisions of the current AASHTO LRFD Bridge Design Specifications. This is a deviation from the AASHTO LRFDLTS-1, which refer the design of welded connections to the AWS D1.1 Structural Welding Code.

For truss superstructures, current practice is to design and provide alternate details of the connection of web members (angles) to main chord members (HSS tubular round sections) for both welded and bolted connections, except the chord to column connection and first panel of cantilever trusses which must be bolted. This affords the fabricator the option of galvanizing individual members prior to truss fabrication (using bolted connections) or galvanizing entire truss segments after assembly (using bolted or welded connections).

39.4.4 OSS Standard Designs

Standard Design OSS types and limitations are listed in Table 39.1-2 and Table 39.1-3. Because these structures are pre-designed and pre-detailed the involvement of a Department structural engineer is usually not required. Bureau of Structures is responsible for maintaining and updating the Standard Designs as needed.

The Standard Design OSS types were developed to cover a wide range of signage requirements while placed over typical roadway and roadside configurations. Standard Designs are not intended to cover unique situations or unusual geometry, or for reasons described in 39.4.5. Contact the Bureau of Structures Design Section with questions regarding applicability of standard designs.

Standard Foundation Designs are included in the OSS Standard Detail Drawings. See 39.5 for further guidance on subsurface investigation, assumed soil parameters, and foundation design.

When Standard Design OSS are used in a project, the Region or their consultant prepare final contract plans as described in 39.1.9. This includes filling out the appropriate sign structure specific information on both the General Notes and General Layout sheets and combining those with the appropriate OSS Standard Design Drawings to make up the final plan set.

39.4.5 OSS Non-Standard Designs

Design and plan detailing must be provided by Bureau of Structures or by a structural design consultant for all non-standard designs. The following circumstances warrant a non-standard design:

1. The OSS type is Butterfly, Butterfly Truss, or Bridge Mounted

2. The OSS type falls outside the limits of span length, sign area, DMS weight, or sign height in FDM 11-55-20 Figure 20.2.3 and Figure 20.2.4.

3. Region soil engineer advises that subsurface conditions at the site are expected to negatively differ from assumed soil profile and design parameters of standard foundations (e.g. soft soil or shallow bedrock – see 39.5.2.2).

4. Excessive sign structure height (e.g. sign structure behind MSE wall) or requires the use of concrete column (designed for impact load – see 39.4.2.2)

BOS must be consulted to verify and confirm the need for individual designs before undertaking this effort.

The design detailing shall generally follow the guidance provided by the OSS Standard Design Drawings but should clearly delineate any required changes to individual member sizes, connections and foundation details necessary to satisfy the AASHTO LRFDLTS-1 Design Specifications.

In some instances, it may still be appropriate to use part or all of the Standard Designs even though the sign structure is considered a Non-standard Design. A couple of examples include:

1. A sign structure has both static and DMS sign types specified for mounting (consult with BOS before using a standard design in this situation).

2. A Standard Design structure is used in conjunction with a Non-standard foundation. See section 39.5.3.

In any case, the sign structure is still considered a Non-standard design in terms of the design process and should proceed as detailed in 39.1.9.

39.4.6 OSS Contractor Designed

Contractor Designed OSS types and limitations are listed in Table 39.1-2 and Table 39.1-3. Because these structures are designed by the contractor, involvement of a Department structural engineer is usually not required.

Standard Foundation Designs are included in the OSS Standard Detail Drawings. See 39.5 for further guidance on subsurface investigation, assumed soil parameters, and foundation design. Bureau of Structures is responsible for maintaining and updating the standard foundation designs that go along with the Contractor Designed OSS types.

These structures are designed for the required actual sign area and configuration, unless future expansion is anticipated, which should be noted and shown on the plans. The required actual sign area, span length, etc. is used to select the appropriate standard foundation from the figure provided in chapter 11-55-20 of the FDM.

When Contractor Designed OSS are used in a project, the Region or their consultant prepare final contract plans as described in 39.1.9. This includes filling out the appropriate sign structure specific information on both the General Notes and General Layout sheets and combining those with the appropriate OSS Standard Design Drawings to make up the final plan set.

39.5 Geotechnical Guidelines

39.5.1 General

For full span and cantilever 4-chord trusses, the typical preferred foundation is comprised of two cylindrical drilled shafts connected by a concrete cross-girder, as detailed in the OSS Standard Design Drawings. The top of the cross-girder is set 3 feet above the highest ground elevation at the foundation. For all other types, the typical preferred foundation is comprised of a single cylindrical drilled shaft directly supporting the column vertical support. Occasionally, some columns are mounted directly on top of modified bridge parapets, pier caps and concrete towers instead of footings.

There are several potential challenges regarding subsurface exploration for OSS foundations:

- The development and location of these structures are typically not known at the onset of the preliminary design stage, when the most subsurface exploration typically occurs. This creates the potential need for multiple drilling mobilizations for the project.

- OSS are often located in areas of proposed fill soils. The source and characteristics of fill soil is unknown at the time of design.

- OSS foundations are often located on the shoulder or median directly adjacent to high-volume roadways. Obtaining boings in these locations typically requires significant traffic control, night work, and working in a potentially hazardous work zone.

- If a consultant is involved in the project, the unknowns associated with these structures in the project scoping stage complicate the consultant contracting process. It is often difficult to determine the need for OSS specific subsurface investigation at the time the consultant contract is normally being scoped. In cases where the need for a specific subsurface investigation is known or anticipated, an assumption must be made regarding the level of subsurface investigation to include in the consultant design contract. Alternatively, a decision can be made to assume use of standard OSS and foundation designs. If the need for specific subsurface investigation is later determined to be necessary, this may require a change to add it to the consultant contract.

39.5.2 Standard Foundations for OSS

39.5.2.1 General

WisDOT has created standard full span and cantilever 4-chord truss designs that include fully designed and detailed drilled shaft foundations as part of the overall standard design. The standard foundation details are incorporated with the OSS Standard Design Drawings for these structures and are available on the BOS website.

Single drilled shaft OSS Standard Design Drawings for use with contractor designed full span and cantilever 2-chord truss and monotube OSS are also available on the BOS website.

WisDOT has no standard foundation design details for alternate foundation types and the selected alternative foundations would be required to be individually designed and reviewed by BOS.

39.5.2.2 Design Parameters Used for Standard Foundation Design

Standard dual and single drilled shaft foundation designs were developed in accordance with applicable requirements of Section 10 of the *AASHTO LRFD Bridge Design Specifications.*

The standard foundation designs are based on the following design parameters:

- Total Unit Weight = 125 pcf
- Granular Soil Profile: Internal Angle of Friction = 24 degrees, or
- Cohesive Soil Profile: Undrained Shear Strength = 750 psf
- Soil and drilled shaft downward resistance factor ϕ =1.0 [1]
- Drilled shaft uplift resistance factor ϕ =0.8 [1]
- Depth of water table assumed 10 feet below the ground surface
- Soil side resistance is considered fully effective to the top of the drilled shaft or top of ground surface, whichever is the lower elevation.

Note 1: Resistance factors per AASHTO 10.5.3.3 assuming the drilled shaft design is governed by the wind load combination which is an Extreme Event load combination.

<u>WisDOT policy item:</u>
Design of standard sign structure foundations assumes soil side resistance is fully effective to the top of the drilled shafts for full span 4-chord OSS foundations and to within 3 feet below the lowest ground surface for all other OSS foundations. This is a deviation from AASHTO 10.8.3.5 1b.

Use of the standard foundations requires that the in-situ soils parameters at the site meet or exceed the assumed soil design parameters noted above. Soil parameters were selected to be sufficiently conservative to cover most sites across the state. Designers should contact the Region Soils Engineer or the Geotechnical Consultant to assist in the evaluation of the subsurface conditions compared to the assumed soil parameters. An assessment can also be made by checking nearby borings and as-built drawings of nearby existing structures, and similar sources. If there is reason to suspect weaker soils or that shallow bedrock is present, OSS specific soil borings should be obtained to confirm in-situ soil properties meet or exceed the assumed parameters used for the standard designs. If these site-specific soil properties do not meet the above minimums, a special individual foundation design will be required using actual soil parameters determined from a subsurface investigation per 39.5.3.

39.5.3 Standard Base Reactions for Non-Standard Foundation Design

There may be instances when a Standard Design sign structure is used in conjunction with a non-standard foundation, for reasons detailed in 39.4.5. Contact Bureau of Structures to obtaining the Standard Design or Contractor Designed sign structure base reactions that were used in developing the standard foundations.

39.5.4 Subsurface Investigation and Information

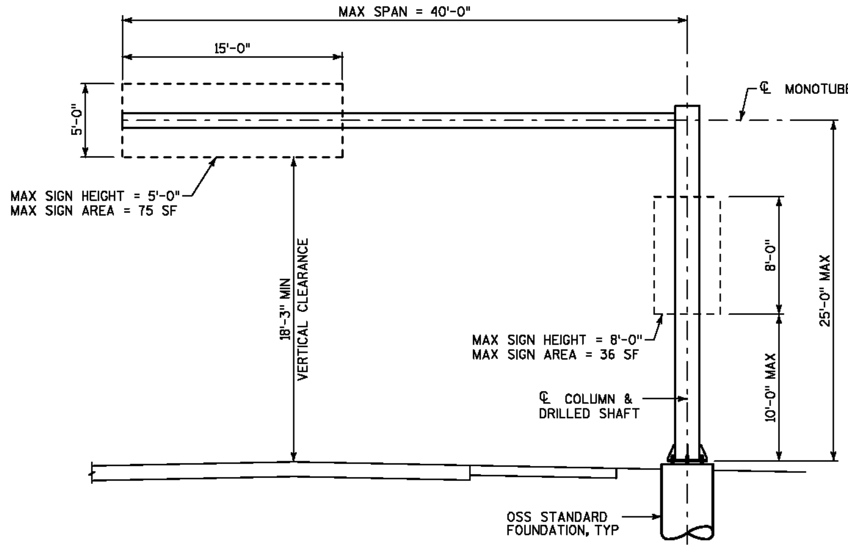
No subsurface investigation/information is necessary for the use of WisDOT standard foundations. Appropriate subsurface information is necessary for any non-standard OSS or situation that is outside any of the standard design ranges of applicability which requires an individual foundation design to be performed.

There may be several methods to obtain the necessary subsurface soil properties for a custom, individual foundation design, as described below:

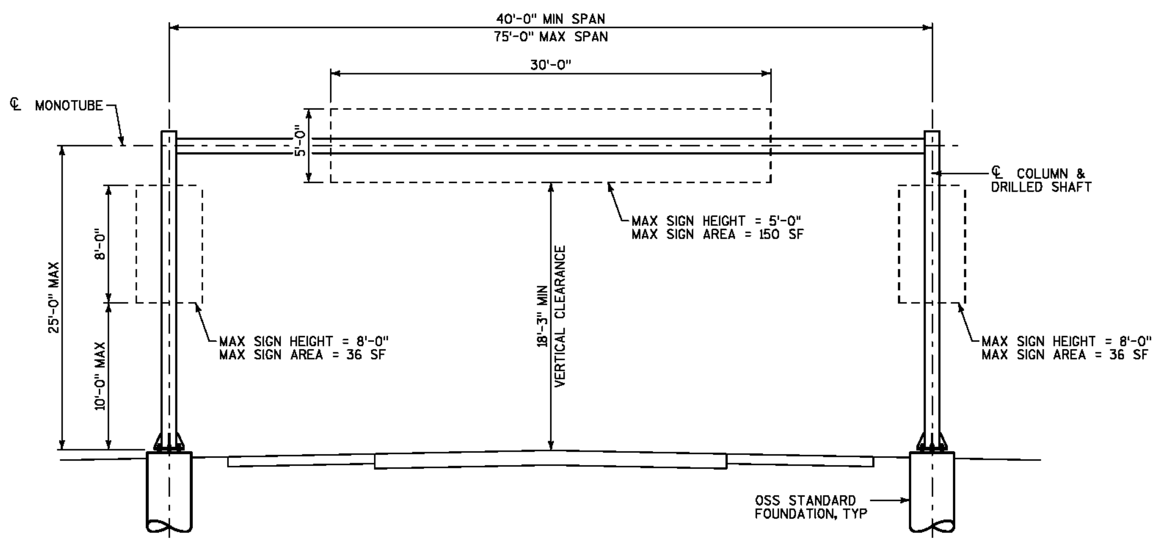
- In areas of fill soils, the borrow material is usually unknown. The designer should use their best judgment as to what the imported soils will be. Standard compaction of this material can be assumed.
- The designer may have a thorough knowledge of the general soil conditions and properties at the site and can reasonably estimate soil design parameters.
- The designer may be able to use information from nearby borings. Judgment is needed to determine if the conditions present in an adjacent boring(s) are representative of those of the site in question.
- If the designer cannot reasonably characterize the subsurface conditions by the above methods, a soil boring and Geotechnical report (Site Investigation Report) should be completed. Necessary soil design information includes soil unit weights, cohesions, phi-angles and location of water table.

Designers, both internal and consultant, should also be aware of the potential of high bedrock, rock fills and the possible conflict with utilities and utility trenches. Conservative subsurface design parameters are encouraged.

39.6 Appendix – OSS Limiting Parameters



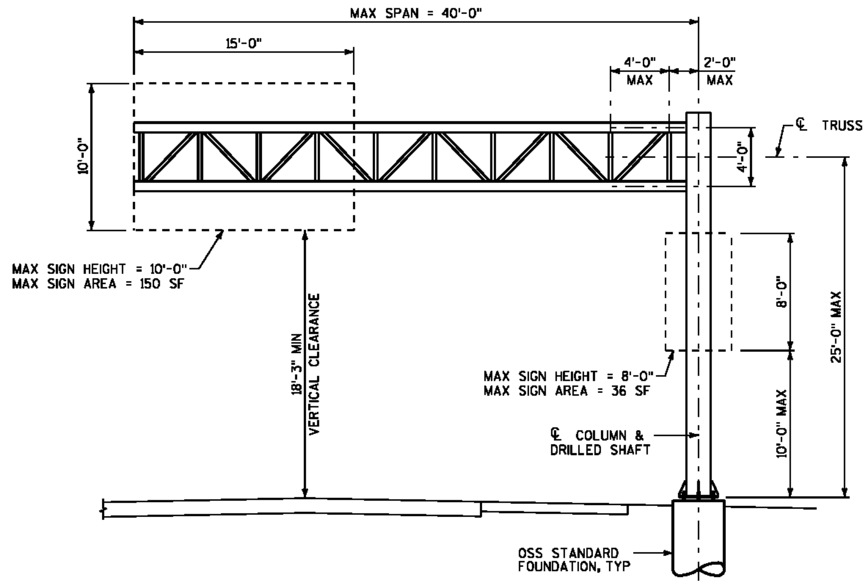
CANTILEVER MONOTUBE



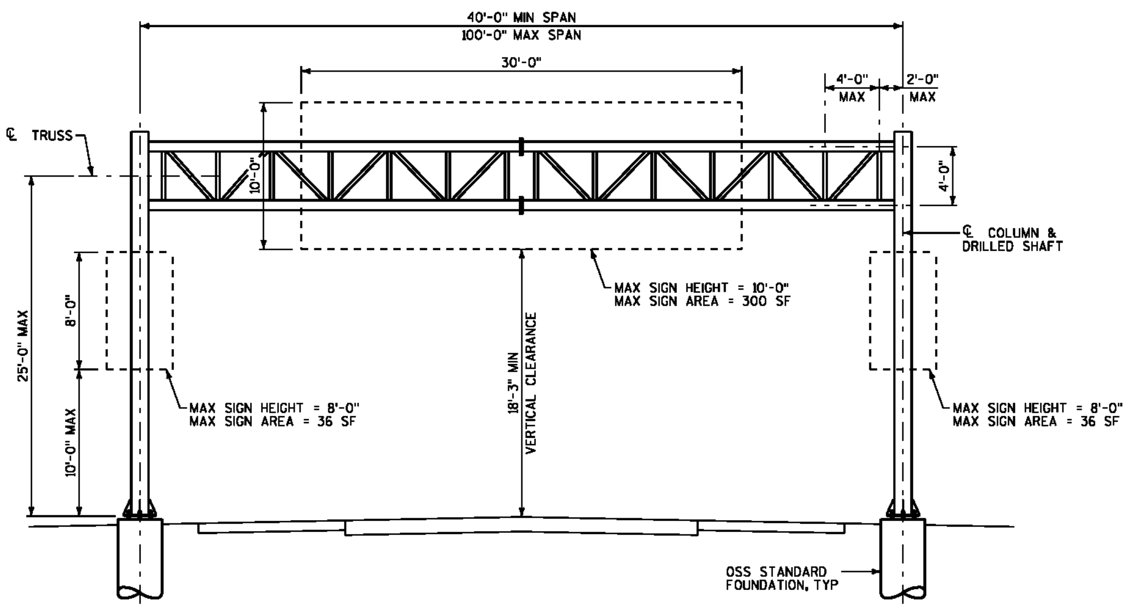
FULL SPAN MONOTUBE

DESIGNER NOTES:

1. SELECT STRUCTURE TYPE BASED ON REQUIRED SPAN LENGTH AND DESIGN SIGN AREA TO BE SUPPORTED ON THE STRUCTURE. SHOW ON THE "GENERAL LAYOUT" SHEET FOR THE STRUCTURE.
2. SELECT OSS STANDARD FOUNDATION TYPE AND SHOW ON THE "GENERAL LAYOUT" SHEET. SEE "OSS MONOTUBE & 2-CHORD TRUSS STANDARD FOUNDATIONS" SHEET AND SECTION 11-55-20 OF THE WISDOT FACILITIES DEVELOPMENT MANUAL FOR FOUNDATION SELECTION CRITERIA.
3. COLUMN SIGN AREA DOES NOT CONTRIBUTE TO THE SELECTION LIMITS OF THE FDM, BUT ARE PERMITTED UP TO THE LIMITS SHOWN IN THE FIGURES.



CANTILEVER 2-CHORD TRUSS



FULL SPAN 2-CHORD TRUSS

DESIGNER NOTES:

1. SELECT STRUCTURE TYPE BASED ON REQUIRED SPAN LENGTH AND DESIGN SIGN AREA TO BE SUPPORTED ON THE STRUCTURE. SHOW ON THE "GENERAL LAYOUT" SHEET FOR THE STRUCTURE.
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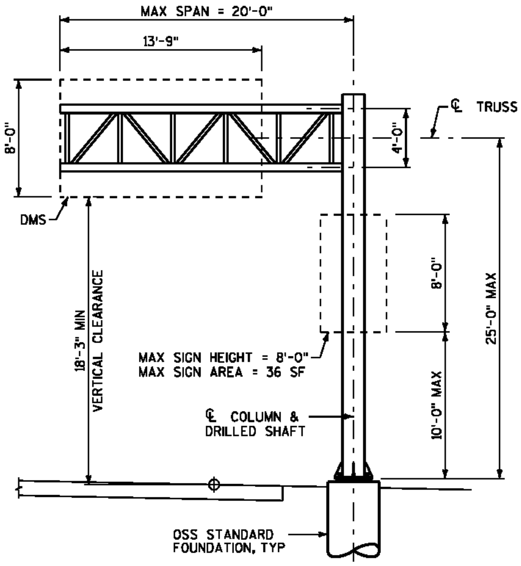
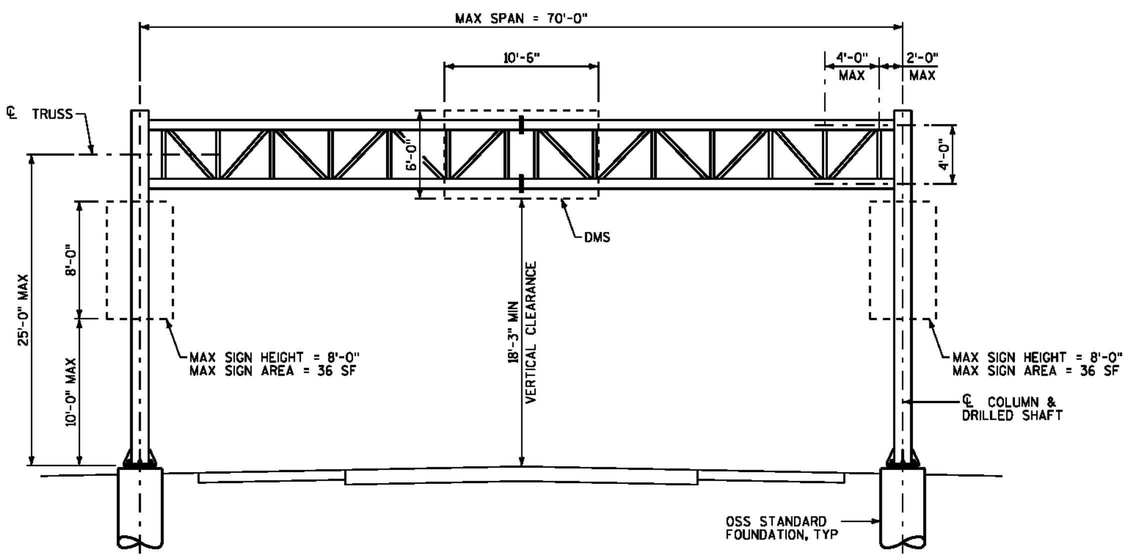


CANTILEVER 2-CHORD TRUSS DMS



FULL SPAN 2-CHORD TRUSS DMS

DESIGNER NOTES:

1. SELECT STRUCTURE TYPE BASED ON REQUIRED SPAN LENGTH AND DESIGN SIGN AREA TO BE SUPPORTED ON THE STRUCTURE. SHOW ON THE "GENERAL LAYOUT" SHEET FOR THE STRUCTURE.
2. SELECT OSS STANDARD FOUNDATION TYPE AND SHOW ON THE "GENERAL LAYOUT" SHEET. SEE "OSS MONOTUBE & 2-CHORD TRUSS STANDARD FOUNDATIONS" SHEET AND SECTION 11-55-20 OF THE WISDOT FACILITIES DEVELOPMENT MANUAL FOR FOUNDATION SELECTION CRITERIA.
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Table of Contents

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.

40.2 History

40.2.1 Concrete

Prior to 1975, all concrete structures were designed by the Service Load Method. The allowable design stress was 1400 psi based on an ultimate strength of 3500 psi except for deck slabs. In 1965, the allowable stress for deck slabs on stringers was reduced from 1400 to 1200 psi. The reason for the change was to obtain thicker concrete decks by using lower strength. The thicker deck was assumed to have more resistance to cracking and greater durability. During this time no changes were made in the physical properties of the concrete or the mixing quantities until 1974 when more cement was added to the mix for concrete used in bridge decks.

In 1980, the use of set retarding admixtures was required when the atmospheric temperature at the time of placing the concrete is 70°F or above; or when the atmospheric temperature is 50°F or above and it is expected that the time to place the concrete of any span or pour will require four or more hours. Retarding admixtures reduce concrete shrinkage cracking and surface spalling. Also, during the early 1980's a water reducing admixture was provided for Grades A and E concrete mixes to facilitate workability and placement.

40.2.2 Steel

Prior to 1975, Grade 40 bar steel was used in the design of reinforced concrete structures. This was used even though Grade 60 bars may have been furnished on the job. Allowable design stress was 20 ksi using the Service Load Method and 40 ksi using the Load Factor Method.

40.2.3 General

In 1978, Wisconsin Department of Transportation discovered major cracking on a two-girder, fracture critical structure, just four years after it was constructed. In 1980, on the same structure, major cracking was discovered in the tie girder flange of the tied arch span.

This is one example of the type of failures that transportation departments discovered on welded structures in the 1970's and '80's. The failures from welded details and pinned connections led to much stricter standards for present day designs.

All areas were affected: Design with identification of fatigue prone details and classifications of fatigue categories (AASHTO); Material requirements with emphasis on toughness and weldability, increased welding and fabrication standards with licensure of fabrication shops to minimum quality standards including personnel; and an increased effort on inspection of existing bridges, where critical details were overlooked or missed in the past.

Wisconsin Department of Transportation started an in-depth inspection program in 1985 and made it a full time program in 1987. This program included extensive non-destructive testing. Ultrasonic inspection has played a major role in this type of inspection. All fracture critical structures, pin and hanger systems, and pinned connections are inspected on a 72-month cycle.

40.2.4 Funding Eligibility and Asset Management

Nationally, MAP-21 (2012) and the FAST Act (2015) have moved structures asset management to a more data-driven approach. Funding restrictions with regards to Sufficiency Rating, Structural Deficiency, and Functional Obsolescence have been removed or significantly revised. In place of these past restrictions, MAP-21 requires the development and approval of a statewide Transportation Asset Management Plan (TAMP). A key part of the WisDOT TAMP is the Wisconsin Structures Asset Management System (WiSAMS).

WiSAMS is being developed as a planning tool, which analyzes current structure inspection data, projects future deteriorated structure condition, and applies the Bridge Preservation Policy Guide (BPPG) to recommend appropriate structure work actions at the optimal time. WiSAMS is a tool for regional and statewide programming, and is not designed as an in-depth scoping tool. WiSAMS may provide an estimate of the appropriate work action, but an in-depth evaluation of the actual structure condition and appropriate scope of work (SSR) and consideration of other non-structural project factors (e.g. cost and functionality) is still required.

In Wisconsin, the Local Bridge Program, through State Statute 84.18 and Administrative Rule Trans 213, is still tied to historic FHWA classifications of Sufficiency Rating, Structural Deficiency, and Functional Obsolescence.

The most effective way to reduce the amplitude of traffic-induced vibrations is to maintain a smooth structure approach and riding surface. Wisconsin has experienced some cracking in the concrete overlays and parapets during rehabilitation construction. The apparent cause of cracking is related to traffic impact due to rough structure approaches. Particular attention should be given to providing a smooth transition on temporary approaches and over expansion joints on all bridge decks during rehabilitation. Other causes of cracking are related to shrinkage, temperature, wind induced vibrations, concrete quality and improper curing.

Options for deck rehabilitation are as follows:

1. Asphalt Patch
2. Asphalt or Polymer Modified Asphaltic Overlay
3. Concrete or Modified Concrete Patch
4. Waterproof Membrane with an Asphalt Overlay (currently not used)
5. Concrete Overlay - Grade E Low Slump Concrete, or Micro-Silica Modified Concrete

Consider the following criteria for rehabilitation of highway bridges:

1. Interstate Bridges as Stand Alone Project
    a. Deck condition equal 4 or 5 and;
    b. Wear course or wear surface less than or equal to 3.
    c. No roadway work scheduled for at least 3 years.
2. Interstate Bridge with Roadway Work
    a. Deck Condition less than or equal 4.
    b. Wear course or wear surface less than or equal to 4.
3. Rehab not needed on Interstate Bridges if:
    a. Deck condition greater than 4.
    b. Wear surface or wear course greater than or equal 4.
4. All Bridges

WisDOT policy item:

On major rehab work, build to current standards such as safety parapets, full shoulder widths, etc. Use the current Bridge Manual standards and tables. Exceptions to this policy require approval from the Bureau of Structures Development Section.

a. Evaluate cost of repeated maintenance, traffic control as well as bridge work when determining life-cycle costs.

b. Place overlays on all concrete superstructure bridges if eligible.

c. For all deck replacement work the railing shall be built to current standards.

5. All Bridges with Roadway Work

   Coordinate with the Region the required staging of bridge related work.

A number of specific guidelines are defined in subsequent sections. As with any engineering project, the engineer is allowed to use discretion in determining the applicability of these guidelines.

40.9 Superstructure Replacement

Various types of superstructure replacements include replacing prestressed girders in-kind, replacing slabs in-kind and replacing steel girders with prestressed girders or slabs. When considering replacement of a deck on steel girders, consideration of the cost of painting the structural steel should be included in the evaluation.

Approval is required from BOS for all superstructure replacement projects. To ensure that the cost of a superstructure replacement is warranted, the substructure should be in good condition. In general, the superstructure replacement should remain the same as the original design to better ensure that substructure reuse is practical. See 40.10 for considerations regarding substructure reuse criteria.

WisDOT policy item:

Provided that the substructure meets the criteria in 40.10, the superstructure may be replaced. The superstructure shall be designed to current LRFD criteria.

Reuse of the existing substructure is contingent on the fixity of the substructure units remaining the same. If the fixity is changed, the substructure must be evaluated per the design loading of the original structure.

With the substructure needing further evaluation for increased dead load and/or change in fixity, discuss with BOS the acceptability of the evaluation results prior to continuing with final design.

40.10 Substructure Reuse and Replacement

When practical, substructure reuse may be an acceptable alternative to replacing the entire bridge. However, reuse will require early coordination with BOS, engineering judgement, and will be evaluated on a project-by-project basis. This evaluation should determine if the substructure can be reused “as-is” with or without minor surface repairs, reused with major repairs and/or strengthening, or needs to be replaced.

In general, “as-is” reuse of substructures should be reserved for in-kind superstructure replacements with little to no change in geometry, fixity, and service dead loads. Additionally, substructures should be in good condition and only require minor surface repairs. If satisfied, evaluation of the existing substructure with the load rating methodology as discussed in 45.3.2 for an existing (in-service) bridge (e.g. LFR) may be acceptable. An example of this condition would be an in-kind slab superstructure replacement with a substructure that remains in good condition. For other conditions (i.e. reuse with major repairs and/or strengthening), the substructure should be evaluated with the current load rating methodology (LRFR) as discussed in 45.3.1.1 for new bridge construction. If substructure reuse is found to be not practical due the expensive repairs and/or excessive strengthening, the substructure should be completely replaced.

Approval is required from BOS for all substructure reuse projects.

Normally it is acceptable to assume that the original bridge design was done correctly, however pier caps, either for multi-columned piers or open pile bents, have occasionally been under-designed. Further investigation is warranted for pier caps with nominal shear stirrups, rather than stirrups that appear to be designed for the girder configuration, etc.

See 40.15 for more information on substructure inspection.

Additional guidance regarding substructure reuse can be found in the FHWA publication *Foundation Reuse for Highway Bridges*.

40.10.1 Substructure Rehabilitation

Substructure rehabilitation work can vary significantly from minor concrete surface repairs to major repairs that includes strengthening members.

40.10.1.1 Piers

Pier caps and/or columns/shafts may show signs of distress due to spalled concrete. The spalling may be completely around some of the longitudinal bar steel, thus destroying the bond. The concrete usually remains sound under the bearing plates, possibly due to compressive forces preventing salt intrusion and/or deterioration from freeze thaw cycles.

If the bond of the structural reinforcement is not compromised (at least half of the bar is bonded), rehabilitation measures include:

1. Concrete surface repair for smaller areas. Fiber Reinforced Polymer (FRP), either non-structural or structural, may be required. See 40.20 for more information on FRP.

2. Column encapsulation. Even if the bars are bonded, the encapsulation provides protection to further damage from snow impact produced by plowing. For encapsulation:

    a. Place adhesive anchors

    b. Place wire mesh around column

    c. Pour 6” concrete encapsulation

If the bond of the structural reinforcement is compromised (at least half of the bar is not bonded), rehabilitation measures include:

1. Cap and/or column/shaft encapsulation. Fiber Reinforced Polymer (FRP), either non-structural or structural, may be required. See 40.20 for more information on FRP.

2. Column encapsulation. The encapsulation provides protection to further damage from snow impact produced by plowing. For encapsulation:

    a. Place adhesive anchors

    b. Place wire mesh around column

    c. Pour 6” concrete encapsulation

40.10.1.2 Bearings

Bearings being replaced should follow the Chapter 27 Standard Details, as well as the Chapter 40 Standard for Expansion Bearing Replacement Details. Replace lubricated bronze bearings with either laminated elastomeric bearings (preferred, if feasible) or Stainless Steel TFE bearings. If only outside bearings are replaced, the difference in friction/resistance values between adjacent girders can be ignored. In addition to the bid item for the new bearing, the STSP Removing Bearings is required.

For bearings requiring maintenance, consider the SPV Cleaning and Painting Bearings. Special Provisions Bearing Maintenance and Bearing Repair may also be worthy of consideration.

40.11 Other Considerations

40.11.1 Replacement of Impacted Girders

When designing a replacement project for girders that have been damaged by vehicular impact, replace the girder in-kind using the latest details. Either HS loading or HL-93 loading may be used if originally designed with non-LRFD methods. Consider using the latest deck details, especially with regard to overhang bar steel.

For the parapet or railing, the designer should match the existing.

40.11.2 New Bridge Adjacent to Existing Bridge

For a new bridge being built adjacent to an existing structure, the design of the new structure shall be to current LRFD criteria for the superstructure and abutment.

The pier design shall be to current LRFD criteria, including the 600 kip impact load for the new bridge. It is not required to strengthen or protect the existing adjacent pier for the 600 kip impact load. However, it would be prudent to discuss with the Region the best course of action. If the Region wants to provide crash protection, it may be desirable to provide TL-5 barrier/crash wall protection for both structures, thus eliminating the need to design the new pier for the 600 kip impact load. The Region may also opt to provide typical barrier protection (< TL-5) to both sets of piers, in which case the design engineer would still be required to design the new pier for the 600 kip impact load. This last option is less expensive than providing TL-5 barrier to both structures. Aesthetics are also a consideration in the above choices.

40.12 Timber Abutments

The use of timber abutments shall be limited to rehabilitation or widening of off-system structures.

Timber abutments consist of a single row of piling capped with timber or concrete, and backed with timber to retain the approach fill. The superstructure types are generally concrete slab or timber. Timber-backed abutments currently exist on Town Roads and County Highways where the abutment height does not preclude the use of timber backing.

Piles in bents are designed for combined axial load and bending moments. For analysis, the assumption is made that the piles are supported at their tops and are fixed 6 feet below the stream bed or original ground line. For cast-in-place concrete piling, the concrete core is designed to resist the axial load. The bending stress is resisted by the steel shell section. Due to the possibility of shell corrosion, steel reinforcement is placed in the concrete core equivalent to a 1/16-inch steel shell perimeter section loss. The reinforcement design is based on equal section moduli for the two conditions. Reinforcement details and bearing capacities are given on the Standard Detail for Pile Details. Pile spacing is generally limited to the practical span lengths for timber backing planks.

The requirements for tie rods and deadmen is a function of the abutment height. Tie rods with deadmen on body piling are used when the height of "freestanding" piles is greater than 12 feet for timber piling and greater than 15 feet for cast-in-place concrete and steel "HP" piling. The "freestanding" length of a pile is measured from the stream bed or berm to grade. If possible, all deadmen should be placed against undisturbed soil.

Commercial grade lumber as specified in AASHTO having a minimum flexural resistance of 1.2 ksi is utilized for the timber backing planks. The minimum recommended nominal thickness and width of timber backing planks are 3 and 10 inches, respectively. If nominal sizes are specified on the plans, analysis computations must be based on the dressed or finished sizes of the timber. Design computations can be used on the full nominal sizes if so stated on the bridge plans. For abutments constructed with cast-in-place concrete or steel "HP" piles, the timber planking is attached with 60d common nails to timber nailing strips which are bolted to the piling.

40.13 Survey Report and Miscellaneous Items

Prior to scheduling bridge plans preparation, a Rehabilitation Structure Survey Report Form is to be completed for the proposed work with field and historical information. The Rehabilitation Structure Survey Report provides the Bridge Engineer pertinent information such as location, recommended work to be performed, and field information needed to accomplish the rehabilitation plans. A brief history of bridge construction date, type of structure, repair description and dates are also useful in making decisions as to the most cost effective rehabilitation. Along with this information, it is necessary to know the extent of deck delamination and current deck, super and substructure condition ratings. A thorough report recommending structural repairs and rehabilitation with corresponding notes is very useful for the designer including information from special inspections.

Because new work is tied to existing work, it is very important that accurate elevations be used on structure rehabilitation plans involving new decks and widenings. Survey information of existing beam seats is essential. Do not rely on original plans or a plan note indicating that it is the responsibility of the contractor to verify elevations.

Experience indicates that new decks open to traffic and subjected to application of de-icing salts within the first year show signs of early deck deterioration. Therefore, a protective surface treatment is applied to all new bridge deck concrete as given in the standard specifications.

For existing sloped faced parapets on decks under rehabilitation, the metal railings may be removed. On deck replacements and widenings, all existing railings are replaced. Refer to Bridge Manual Chapter 30-Railings for recommended railings.

On rehabilitation plans requiring removal of the existing bridge name plates; provide details on the plans for new replacement name plates if the existing name plate cannot be reused. The existing bridge number shall be used on the name plates for bridge rehabilitation projects.

If floor drain removal is recommended, review these recommendations for conformance to current design standards and remove any floor drains not required. Review each structure site for length of structure, grade, and water erosion at the abutments.

Bridge rehabilitation plans for steel structures are to provide existing flange and web sizes to facilitate selecting the proper length bolts for connections. If shear connectors are on the existing top flanges, additional shear connectors are not to be added as welding to the top flange may be detrimental. Do not specify any aluminized paint that will come in contact with fresh concrete. The aluminum reacts severely with the fresh concrete producing concrete volcanoes.

Recommended paint maintenance is determined with assistance from the Wisconsin Structures Asset Management System (WiSAMS), which utilizes information provided by the routine bridge inspections.

Structure plans (using a sheet border with a #8 tab) are required for all structure rehabilitation projects. This includes work such as superstructure painting projects and all overlay projects, including polymer overlay projects. See Chapter 6-Plan Preparation guidance for plan minimum requirements.

Existing steel expansion devices shall be modified or replaced with watertight expansion devices as shown in Bridge Manual Chapter 28-Expansion Devices. If the hinge is repaired, consideration should be given to replacing the pin plates with the pins. Replace all pins with stainless steel pins conforming to ASTM 276, Type S20161 or equal. On unpainted steel bridges, the end 6' or girder depth, whichever is greater, of the steel members adjacent to an expansion joint and/or hinge are required to have two shop coats of paint. The second coat is to be a brown color similar to rusted steel. Exterior girder faces are not painted for aesthetic reasons, but paint the hanger on the side next to the web.

A bridge on a steep vertical grade may slide downhill closing any expansion joint on this end and moving the girders off center from the bearings. One possible corrective action is to block the ends of the girder during the winter when the girders have shortened due to cold temperatures. Continued blocking over a few seasons should return the girders to the correct position. The expansion device at the upslope end may have to be increased in size.

Raised pavement markers require embedment in the travel way surface. As a general rule, they are not to be installed on bridge decks due to future leakage problems if the marker comes out. They shall not be installed thru the membranes on asphalt overlays and not at all on asphaltic decks where the waterproofing is integral with the surface.

40.14 Superstructure Inspection

40.14.1 Prestressed Girders

On occasion prestressed concrete girders are damaged during transport, placement, or as a result of vehicle impact. Damage inspection and assessment procedures are necessary to determine the need for traffic restrictions, temporary falsework for safety and/or strength. Three predominant assessment areas are damage to prestressing strands, damage to concrete, and remaining structural integrity.

Where damage to a girder results in any significant loss of concrete section, an engineering analysis should be made. This analysis should include stress calculations for the damaged girder with comparison to the original design stresses. These calculations will show the loss of strength from the damage.

Assessment of damage based on the loss of one or more prestressing strands or loss of prestress force is given as reason for restriction or replacement of girders. Some of the more common damages are as follows with the recommended maintenance action. However, an engineering assessment should be made on all cases.

1. If cracking and spalling are limited to the lower flange, patching is usually performed. This assessment should be based on calculations that may allow repair.

2. If cracking continues from flange into web, the girder is normally replaced. Findings indicate that sometimes repair-in-place may be the preferred decision.

3. Termini of cracks are marked, and if the cracks continue to grow the girder may be replaced. This is a good method to determine the effect of loads actually being carried.

4. When large areas of concrete are affected, or when concrete within stirrups is fractured, replace. At times it may be more appropriate to repair-in-place than replace.

Repair and/or replacement decisions are based on structural integrity. Load capacity is by far the most important rationale for selection of repair methods. Service load capacity needs to be calculated if repair-in-place is contemplated.

Evaluations of criteria for assessment of damage related to the two predominant areas associated with structural integrity are the following:

1. A structural analysis is made to determine the stresses in a damaged girder on the basis of the actual loads the girder will carry. This technique may result in a girder that has less capacity than the original, but can still safely carry the actual loads. This assessment results in consideration of possible repair-in-place methods which normally are cost effective.

   or

2. A structural analysis is made to determine the load capacity and rating of the girder. If the capacity and rating of the girder is less than provided by the original design, the girder shall be replaced. This assessment will provide a girder equal to the original design, but precludes possible repair-in-place methods that are normally less costly.

Location and size of all spalled and unsound concrete areas shall be recorded. Location, length, and width of all visible cracks shall be documented. All damage to prestress strands and reinforcing steel shall be reported. Location of hold-down devices in the girder shall be shown in relation to the damage. Horizontal and vertical misalignment along the length of the girder, and at points of localized damage, shall be reported. (These measurements might best be made by string-lining). Growth of cracks shall be monitored to determine that the cracked section has closed before extending to the web.

Critical damage is damage to concrete and/or the reinforcing elements of prestressed concrete girders such as:

1. Cracks extend across the bottom flange and/or in the web directly above the bottom flange. (This indicates that the prestressing strands have exceeded yield strength).

2. An abrupt lateral offset is measured along the bottom flange or lateral distortion of exposed prestressing strands. (This also indicates that the prestressing strands have exceeded yield strength).

3. Loss of prestress force to the extent that calculations show that repairs cannot be made.

4. Vertical misalignment in excess of the normal allowable.

5. Longitudinal cracks at the interface of the web and the top flange that are not substantially closed below the surface. (This indicates permanent deformation of stirrups).

40.14.2 Steel Beams

These are three alternate methods of repairing damaged steel beams. They are:

1. Replace the total beam,

2. Replace a section of the beam, or

3. Straighten the beam in-place by heating and jacking.

The first alternate would involve removing the concrete deck over the damaged beam, remove the damaged section and weld in a new piece; then reconstruct the deck slab and railing over the new girder. Falsework support is required at the locations where the beams are cut and probably in the adjacent span due to an unbalanced condition.

The second alternate involves cutting out a section of the beam after placing the necessary supporting members. The support is placed using calibrated jacks. The section is cut out as determined by the damage. A new section plus any vertical stiffeners and section of cover plates would be welded in. This involves butt welds on both the flange and web. The welding of the web is difficult due to minor misalignments to start with plus the tendency of thin plates to move from the heat of welding.

The third alternate of heating and jacking the in-place beam to straighten it is a difficult procedure but can be done by personnel familiar and knowledgeable of the process. It is important to maintain heat control under 1300°F maximum. Use an optical pyrometer to determine heat temperature. There is no specified tolerance for the straightened member. The process is deemed satisfactory when a reasonable alignment is obtained.

Based on the three alternates available, the estimated costs involved and the resultant restoration of the beam to perform its load carrying capacity, heat straightening is a viable option in many cases.

The structural engineer who will be responsible for plan preparation should field review the site with the Regional Bridge Maintenance Engineer.

40.15 Substructure Inspection

The inspection of substructure components may reveal deteriorating concrete in areas exposed to de-icing chemicals from roadway drainage or concrete disintegration where exposed in splash zones or stream flows. Footings and pilings exposed due to erosion and undermining could result in loss of bearing capacity and/or section. Utilize HSIS data to flag potential scour concerns (code 6000), with scour defects in condition state 4 being a significant concern.

Abutment and pier concrete reuse may require core tests to determine the quality and strength of the concrete. Original pile capacities are determined from plans, or if available, the pile driving records. Reuse of steel pile sections will require checking the remaining load carrying capacity if section loss is determined to be present. Steel piling should be checked:

- Immediately below the splash zone or water line for deterioration and possible loss of section. High section loss occurs in some areas due to corrosion from bacterial attack at 3 to 6 feet below the water line.

- Below abutments where the berm soil (material beneath riprap) has settled below the abutment bottom and water appears to be flowing from beneath the abutment or stream water has direct access to the piling.

If there is piling section loss or undermined spread footings, capacities of existing piling and/or footings will need to be recomputed for load rating purposes.

Timber substructure components may exhibit deterioration due to fungus decay, abrasion wear and weathering. Also, physical damage may be caused by vermin attacks, chemicals, fires, and collisions. Prior to reuse, timber backed abutments and pier bents shall be checked, by boring, for material and mechanical condition, section loss and structural adequacy. Generally, timber substructures are not good candidates for substructure reuse due to their limited service life.

Bearing condition needs to be evaluated. When possible, replacement expansion bearings should be laminated elastomeric bearings. Replacing expansion devices to reduce chloride infiltration is often warranted.

40.16 Concrete Anchors for Rehabilitation

Concrete anchors are used to connect concrete elements with other structural or non-structural elements and can either be cast into concrete (cast-in-place anchors) or installed after concrete has hardened (post-installed anchors). This section discusses post installed anchors used on bridge rehabilitation projects. Note: this section is also applicable for several cases where post installed anchors may be allowed in new construction.

This section includes guidance based on the ACI 318-14 manual, hereafter referred to as ACI. (AASHTO currently does not have guidance for anchors.)

40.16.1 Concrete Anchor Type and Usage

Concrete anchors installed in hardened concrete, post-installed anchors, typically fall into two main groups – adhesive anchors and mechanical anchors. For mechanical anchors, subgroups include undercut anchors, expansion (torque-controlled or displacement controlled) anchors, and screw anchors.

Mechanical anchors are seldom used for bridge rehabilitations and current usage has been restricted due to the following concerns: anchor installation (hitting rebar, abandoning holes, and testing), the number of different anchor types, design requirements that are more restrictive than adhesive anchors, the ability to remove and reuse railings/fences, and the collection of salt water within the hole. Note: mechanical anchors may be considered when it has been determined cast-in-place anchors or through bolts are cost prohibitive, adhesive anchors are not recommended, and the above concerns for mechanical anchors have been addressed. See post-installed anchor usage restrictions for additional information.

An Approved Products List addresses some of the concerns for creep, shrinkage, and deterioration under load and freeze-thaw cycles for adhesives anchors. Bridge rehabilitations projects typically use adhesive anchors for abutment and pier widenings. Other bridge rehabilitation applications may also warrant the use of adhesive anchors when required to anchor into existing concrete. Refer to the Standards for several examples of anchoring into existing concrete.

In limited cases, post installed concrete anchors may be allowed for new construction. One application is the allowance for the contractor to use adhesive anchors in lieu of cast-in-place concrete anchors for attaching pedestrian railings/fencing. Refer to Chapter 30 Standards for pedestrian railings/fencing connections.

The following is a list of current usage restrictions for post installed anchors:

<u>Usage Restrictions</u>:

- Pier cap extensions for multi-columned piers require additional column(s) to be utilized. See Chapter 13 – Piers for structural modeling concepts regarding multi-columned piers.

16. Zureick, A.-H., et al. 2010. *Recommended Guide Specification for the Design of Externally Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements.* Washington D.C.: Transportation Research Board, 2010. NCHRP Report 655

17. Belarbi, A., et al. 2011. *Design of FRP Systems for Strengthening Concrete Girders in Shear.* Washington D.C.: Transportation Research Board, 2011. NCHRP Report 678.

18. Agrawal, A.K., Jalinoos, F., Davis, N., Hoomaan, E., Sanayei, M. (2018). *Foundation Reuse for Highway Bridges,* FHWA-HIF-18-055, Federal Highway Administration, Washington, D.C.

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then C_M shall be 1.0. For large glulam girders covered with deck and wearing surface in good condition such that the girders remain dry, C_M = 1.0.

- Beam Stability (C_L): All girders with decks fastened in the normal manner shall be assumed to have continuous lateral stability and C_L shall be 1.0. If the girders are not prevented from rotating at the points of bearing, or rating engineer determines that there is not continuous lateral support on the compression edge, C_L shall be determined by **NDS [3.3.3]**.

- Size (C_F): Bending stresses for sawn lumber shall be multiplied by the appropriate factor per the footnotes in NDS.

- Volume (C_v): Bending stresses for glued laminated timber shall be multiplied by the appropriate factor per the footnotes in NDS.

- Flat Use (C_{fu}): Bending stresses shall be multiplied by the appropriate factor per NDS, for plank decking loaded on the wide face.

- Repetitive Member (C_r): Bending stresses shall be multiplied by 1.15 on longitudinal nail laminated bridges and on nail laminated decks. For deck planks, 1.15 may be used if they are covered by bituminous surface or perpendicular planks for load distribution and are spaced not more than 24" on center.

- Condition Treatment Factor (C_{pt}): Piling, Bending, Shear, and Compression stresses shall be multiplied by: 1.0 for all douglas fir pile installed prior to 1985, and by 0.9 for all other piles.

- Load Sharing Factor (C_{ls}): This shall be typically be 1.0 for all bents. A higher value may be used per **NDS [6.3.11]** when multiple piles are connected by concrete caps or equivalent force distributing elements so that the pile group deforms together.

- Column Stability (C_P): Compression stresses in bents shall by multiplied by C_p per **NDS [3.7]**. "d" in the formula shall be the minimum measured remaining pile dimension. Unless determined otherwise by the rating engineer, it shall be assumed that all the piles shall have the area and C_p of the worst pile.

The adjusted allowable stress used in ratings shall be the given stress multiplied by all the applicable adjustment factors.

45.6 WisDOT Load Rating Policy and Procedure – Superstructure

This section contains WisDOT policy items or guidance related to the load rating of various types of bridge superstructures.

45.6.1 Prestressed Concrete

For bridges designed to be continuous over interior supports, the negative capacity shall come from the reinforcing steel in the concrete deck. Conservatively, only the top mat of steel deck reinforcing steel should be considered when rating for negative moment. If this assumption results in abnormally low ratings for negative moment, contact the Bureau of Structures Rating Unit for consultation.

Elastic gains in prestressed concrete elements shall be neglected for a conservative approach.

Shear design equations for prestressed concrete bridges have evolved through various revisions of the AASHTO design code. Because of this, prestressed concrete bridges designed during the 1960s and 1970s may not meet current shear capacity requirements. Shear capacity should be calculated based on the most current AASHTO code, either LFR or LRFR. Shear should be considered when determining the controlling ratings for a structure. If shear capacities are determined to be insufficient, the load rating engineer of record should contact the Bureau of Structures Rating Unit for consultation. If an existing bridge was designed using the Simplified Procedure for shear, the Simplified Procedure **LRFD [5.8.3.4.3]** (7^{th} Edition - 2014) may be considered for shear ratings.

If an option is given on the structure plans to use either stress relieved or low relaxation strand, or $^{7}/_{16}$" or $^{1}/_{2}$" diameter strand, consult the shop drawings for the structure to see which option was exercised. If the shop drawings are not available, all possible options should be analyzed and the option which gives the lowest operating rating should be reported.

45.6.1.1 I-Girder

Bridges may have varying girder spacing between spans. A historically common configuration in Wisconsin for prestressed I-girder superstructures is a four-span bridge with continuous girders in spans 2 & 3 and different (wider) girder spacing in spans 1 & 4 (Note: this configuration is not recommended for new structures). Since the girders don't align, the bridge would need to be rated as three separate units – single span, two-span and single span.

When the shear failure plane crosses multiple stirrup zones, guidance given in the **MBE [6A.5.8]** should be followed to determine an average shear reinforcement area per unit length existing within the shear failure plane. The shear failure plane is assumed to cross through mid-depth of the section with a 45-degree angle of inclination.

It is common practice to use the average haunch height in order to locate the concrete deck in relation to the top of the girder. It is also acceptable to use the actual, precise haunch thicknesses, if they are known. Absent information on the depth of the haunch, 1 ¼" may be assumed. The area of the haunch may be used in calculating section properties, but it is common practice to conservatively ignore for purpose of section properties (haunch dead load must be taken into account). Appropriate consideration of the haunch is the responsibility of the load rating engineer.

45.6.1.1.1 Variable Girder Spacing (Flare)

Girder spacings may vary over the length of a given girder (flared girder configuration). Some analysis software may allow for a varying distribution factor along the length of the girder. This is the most accurate and thus preferred method for dealing with a flared girder layout.

Alternatively, conservative assumptions may be made regarding the live load distribution and the assigned dead load to the girder being analyzed. The rating engineer is responsible for determining the appropriate assumptions and for ensuring that they produce conservative results. The methods described in **LRFD [C4.6.2.2.1]** are acceptable. All assumptions made shall be clearly noted in the calculations and in the load rating summary sheet (See 45.9.1).

45.6.1.2 Box and Channel Girders

For adjacent prestressed box and channel girders, the concrete topping may be considered structural when rebar extends from the girders up into the concrete topping.

45.6.2 Cast-in-Place Concrete

45.6.2.1 Slab (Flat or Haunched)

When using Load Factor Rating (LFR) or Allowable Stress Rating (ASR) and calculating the single lane load distribution factor for concrete slab bridges, the distribution width, E, shall be taken as 12'-0".

Some concrete slab bridges may have been designed with an integral concrete pier cap. This would take the form of increased transverse reinforcement at the pier, most likely combined with a haunched slab design. It is WisDOT experience that the integral pier cap will very rarely control the load ratings and a specific evaluation is not required. However, if the pier cap shows signs of distress, a more detailed load rating evaluation may be required. Consult the Bureau of Structures Load Rating Unit in these cases.

45.6.3 Steel

Consistent with the WisDOT policy item in 24.6.10, moment redistribution should not be considered as a part of the typical rating procedure for a steel superstructure. Moment redistribution may be considered for special cases (to avoid a load posting, etc.). Contact the Bureau of Structures Rating Unit with any questions on the use of moment redistribution.

Plastic analysis shall be used for steel members as permitted by AASHTO specifications, including (but not limited to) Article 6.12.2 (LRFR) and Articles 10.48.1, 10.53.1.1, and 10.54.2.1 (LFR). Plastic analysis shall not be used for members with significant deterioration. Per code, sections must be properly braced in order to consider plastic capacity. For questions on the use of plastic analysis, contact the Bureau of Structures Rating Unit.

If there are no plans for a bridge with a steel superstructure carrying a concrete deck, it shall be assumed to be non-composite for purposes of load rating unless there is sufficient documentation to prove that it was designed for composite action and that shear studs or angles were used in the construction.

When performing a rating on a bridge with a steel superstructure element (deck girder, floorbeam, or stringer) carrying a concrete deck, the element should be assumed to have full composite action if it was designed for composite action and it has shear studs or angles that are spaced at no more than 2'-0" centers.

Steel girder bridges in Wisconsin have not typically been designed to use the concrete deck as part of a composite system for negative moment. A typical design will show a lack of composite action in the negative moment regions (i.e., no shear studs). However, if design drawings indicate that the concrete deck is composite with the steel girder in negative moment regions, the negative moment steel in the concrete deck shall conservatively consist of only the top mat of steel over the piers.

For steel superstructures, an additional dead load allowance should be made to account for miscellaneous items such as welds, bolts, connection plates, etc., unless these items are all specifically accounted for in the analysis. See 24.4.1.1 for guidance on this additional dead load allowance. Alternatively, the actual weight of all the miscellaneous items can be tabulated and added to the applied dead load.

WisDOT policy items:

When load rating in-service bridges, WisDOT does not consider the overload limitations of Section 10.57 of the AASHTO Standard Specification.

45.6.3.1 Fatigue

For structures originally designed using LRFD, fatigue shall not be part of the rating evaluation.

For structures originally designed using ASD or LFD, fatigue ratings shall not be reported as the controlling rating. However, a fatigue evaluation may be considered for load ratings accompanying a major rehabilitation effort, if fatigue-prone details (category C or lower) are present. Fatigue detail categories are provided in **LRFD Table [6.6.1.2.3-1]**. Contact WisDOT Bureau of Structures Rating Unit on appropriate level of effort for any fatigue evaluation.

45.6.3.2 Rolled I-Girder, Plate Girder, and Box Girder

Application of the lever rule in calculating distribution factors for exterior girders may be overly conservative in some short-span steel bridges with closely spaced girders and slab overhangs. In this case, the live load bending moment for the exterior girder may be determined by applying the fraction of a wheel line determined by multiplying the value of the interior stringers or beams by:

W_e/S, where:

W_e = Top slab width as measured from the outside face of the slab to the midpoint between the exterior and interior stringer or beam. The cantilever dimension of any slab extending beyond the exterior girder shall not exceed S/2, measured from the centerline of the exterior beam.

S = Average stringer spacing in feet.

Alternately, live load distribution for this case may be determined by refined methods of analysis or with consideration of lane stripe placement as described in 45.5.1.2.

It is common practice to use the average haunch height in order to locate the concrete deck in relation to the top of the girder. It is also acceptable to use the actual, precise haunch thicknesses, if they are known. Absent information on the depth of the haunch, 1 ¼" may be assumed. The area of the haunch may be used in calculating section properties, but it is common practice to conservatively ignore for purpose of section properties (haunch dead load must be taken into account). Appropriate consideration of the haunch is the responsibility of the load rating engineer.

45.6.3.2.1 Curvature and/or Kinked Girders

The effects of curvature shall be considered for all curved steel girder structures. For structures meeting the criteria specified in **LRFD [4.6.1.2.4]** or the **Curved Steel Girder Guide Specification [4.2]**, the structure may be analyzed as if it were straight. However, regardless of the degree of curvature, the effects of curvature on flange lateral bending must always be considered in the analysis, either directly through a refined analysis or through an approximate method as detailed in **LRFD [C4.6.1.2.4b]** or the **Curved Steel Girder Guide Specification [4.2.1]**. This is applicable to discretely braced flanges. If a flange is continuously braced (e.g. encased in concrete or anchored to deck by shear connectors) then it need not be considered. In determining the load rating, flange lateral bending stress shall be added to the major axis bending flange stress, utilizing the appropriate equations specified in LRFD. When using the Curved Steel Girder Guide Specification, flange lateral bending stress reduces the allowable flange stress.

45.6.3.2.2 Skew

Load rating of steel structures with discontinuous cross-frames, in conjunction with skews exceeding 20 degrees shall consider flange lateral bending stress, either directly through a refined analysis or using approximate values provided in **LRFD [C6.10.1]**. This requirement only applies to structures with multi-member cross frames (X or K-brace), and full depth diaphragms between girders. Flange lateral bending stress is most critical when the bottom flange is stiffened transversely (discretely braced). For structures with shorter single member diaphragms (e.g. C or MC-shapes) between girders, where the bottom flange is less restrained, the load rating need not consider flange lateral bending stress due to skew.

Flange lateral bending stress, whether due to skew or curvature, is handled the same in a load rating equation. Refer to the flange lateral bending discussion in 45.6.3.2.1 for more information.

45.6.3.2.3 Variable Girder Spacing (Flare)

Girder spacings may vary over the length of a given girder (flared girder configuration). Some analysis software may allow for a varying distribution factor along the length of the girder. This is the most accurate and thus preferred method for dealing with a flared girder layout.

Alternatively, conservative assumptions may be made regarding the live load distribution and the assigned dead load to the girder being analyzed. The rating engineer is responsible for determining the appropriate assumptions and for ensuring that they produce conservative results. The methods described in **LRFD [C4.6.2.2.1]** are acceptable. All assumptions made should be clearly noted in the calculations and in the load rating summary sheet (See 45.9.1).

If the girders are flared such that the ratio of change in girder spacing to span length is greater than or equal to 0.015, then a refined analysis may be required. Consult the Bureau of Structures Rating Unit for structures that meet this criteria.

45.6.3.3 Truss

45.6.3.3.1 Gusset Plates

WisDOT requires gusset plates to be load rated anytime the loads applied to a structure are altered (see 45.3). Gusset plates should also be evaluated with reports of any significant deterioration. Rating procedures shall follow those specified in the AASHTO MBE.

45.6.3.4 Bascule-Type Movable Bridges

Apply twice the normal dynamic impact factor to live loading of the end floorbeam per **AASHTO LRFD Movable Spec [2.4.1.2.4]**. The end floorbeam will likely control the load rating of bascule bridges built before 1980.

45.6.4 Timber

As a material, timber is unique in that material strengths are tied to the load rating methodology used for analysis (typically ASD or LRFR for timber). Because of this and because of the fact that design/rating specifications have changed through the years, the load rating engineer must carefully consider the appropriate material strengths to use for a given member. When referencing historic plans, WisDOT recommends using the plans to determine the type of material (species and grade), but then using contemporary sources (including tables in 0) to determine material strengths and for rating methodology.

Based on experience, WisDOT recommends evaluating timber superstructures for posting vehicles when the rating factor falls below 1.25 instead of the typical 1.0. See 45.10 for more information on load posting.

45.6.4.1 Timber Slab

For longitudinal nail laminated slab bridges, the wheel load shall be distributed to a strip width equal to:

(wheel width) + 2x(deck thickness).

On bridges that are showing lamination slippage, breakage on the underside, or loose stiffener beam connections, the strip width shall be reduced to:

(wheel width) + 1x(deck thickness).

45.7 WisDOT Load Rating Policy and Procedure – Substructure

45.7.1 Timber Pile Abutments and Bents

Any decay or damage will result in the reduction of the load-carrying capacity based on a loss of cross-sectional area (for shear and compression) or in a reduction of the section modulus (for moment). The capacity of damaged timber bents will be based on the remaining cross-sectional area of the pile and the column stability factor (C_p) using “d”, the least remaining dimension of the column. Such reductions will be determined by the rating engineer based on field measurements, when available.

Timber piles with significant deterioration and/or tipping shall be load rated with consideration of lateral earth pressure and redundancy. Piles shall be assumed to be fixed 6 feet below the stream bed or ground line and pinned at their tops.