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2.1 Organizational Charts

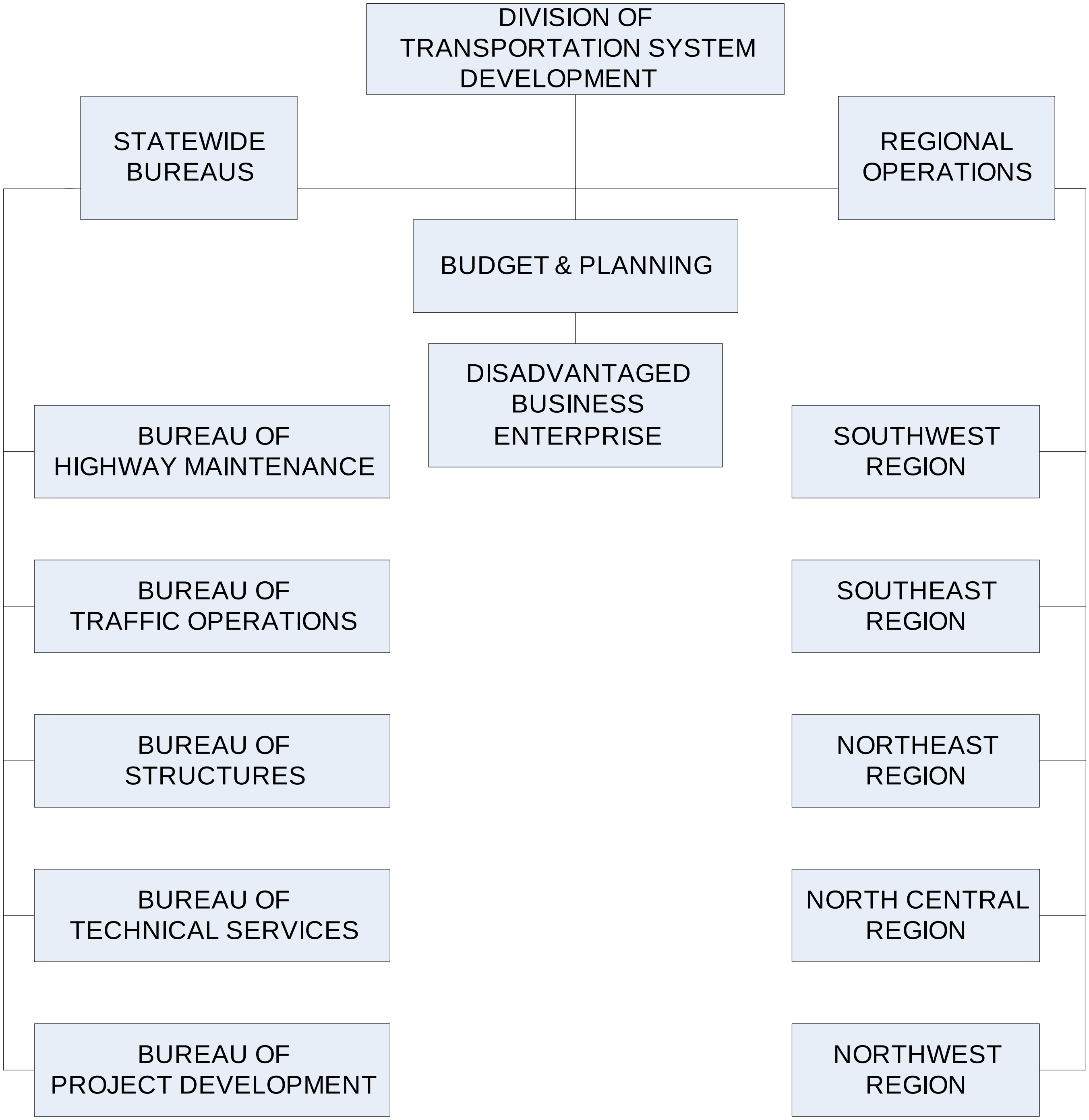


Figure 2.1-1
Division of Transportation System Development

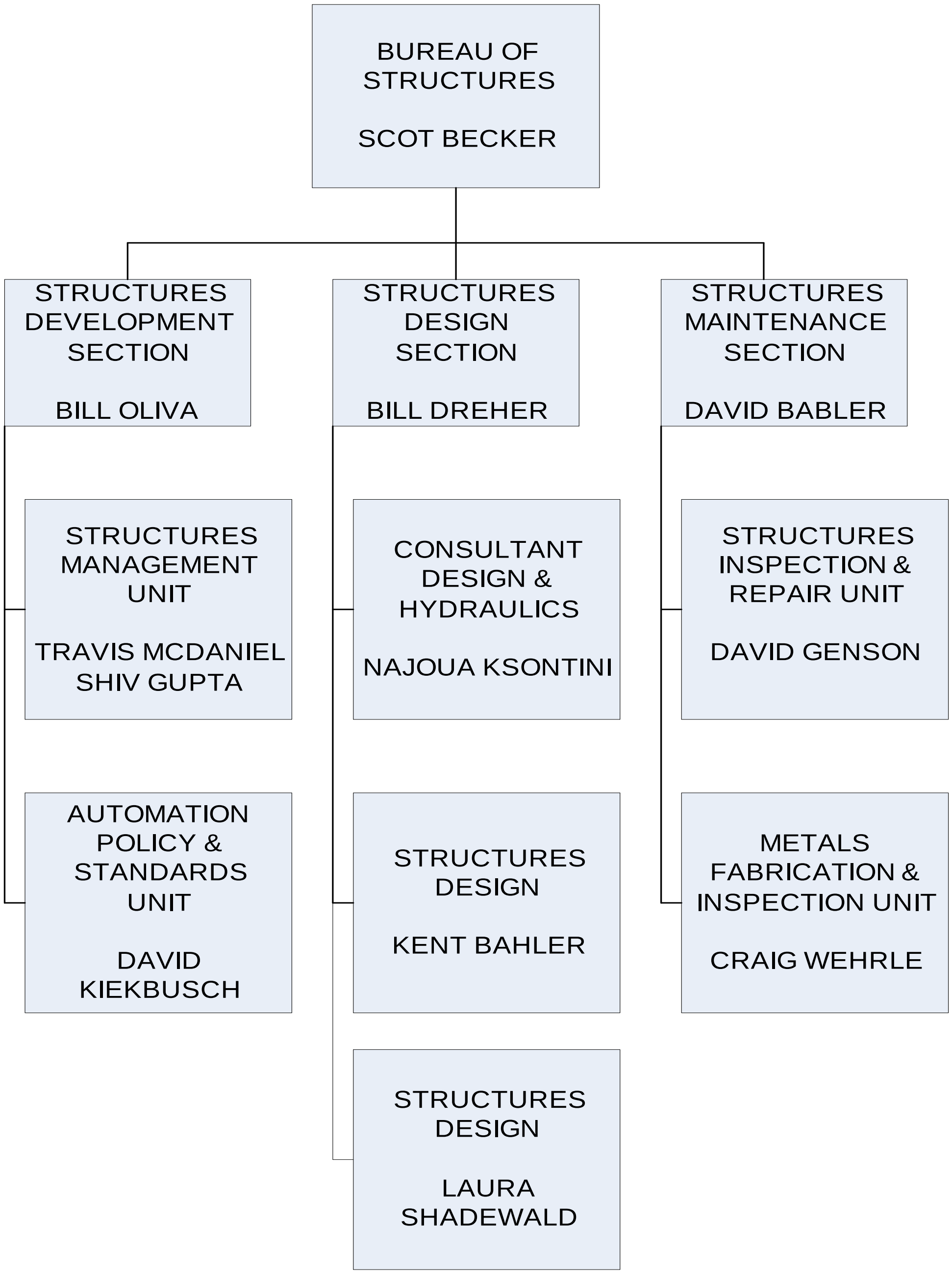
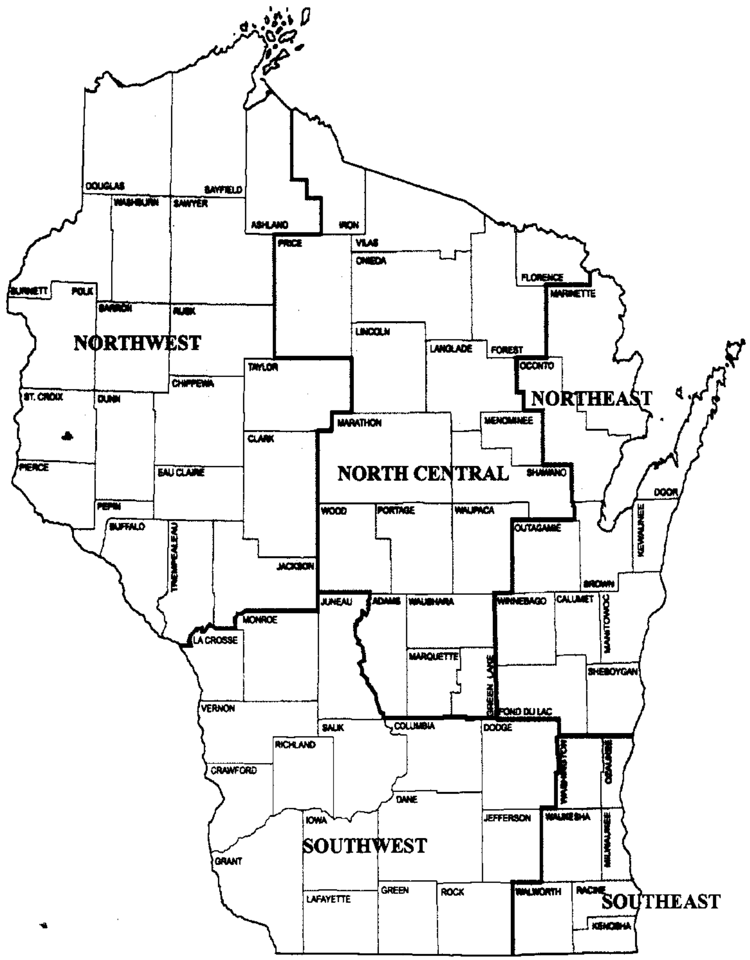


Figure 2.1-2
Bureau of Structures



NO.	COUNTY	REGION	NO.	COUNTY	REGION
1	ADAMS	NORTH CENTRAL	37	MARATHON	NORTH CENTRAL
2	ASHLAND	NORTHWEST	38	MARINETTE	NORTHEAST
3	BARRON	NORTHWEST	39	MARQUETTE	NORTH CENTRAL
4	BAYFIELD	NORTHWEST	40	MILWAUKEE	SOUTHEAST
5	BROWN	NORTHEAST	41	MONROE	SOUTHWEST
6	BUFFALO	NORTHWEST	42	OCONTO	NORTHEAST
7	BURNETT	NORTHWEST	43	ONEIDA	NORTH CENTRAL
8	CALUMET	NORTHEAST	44	OUTAGAMIE	NORTHEAST
9	CHIPPEWA	NORTHWEST	45	OZAUKEE	SOUTHEAST
10	CLARK	NORTHWEST	46	PEPIN	NORTHWEST
11	COLUMBIA	SOUTHWEST	47	PIERCE	NORTHWEST
12	CRAWFORD	SOUTHWEST	48	POLK	NORTHWEST
13	DANE	SOUTHWEST	49	PORTAGE	NORTH CENTRAL
14	DODGE	SOUTHWEST	50	PRICE	NORTH CENTRAL
15	DOOR	NORTHEAST	51	RACINE	SOUTHEAST
16	DOUGLAS	NORTHWEST	52	RICHLAND	SOUTHWEST
17	DUNN	NORTHWEST	53	ROCK	SOUTHWEST
18	EAU CLAIRE	NORTHWEST	54	RUSK	NORTHWEST
19	FLORENCE	NORTH CENTRAL	55	ST CROIX	NORTHWEST
20	FOND DU LAC	NORTHEAST	56	SAUK	SOUTHWEST
21	FOREST	NORTH CENTRAL	57	SAWYER	NORTHWEST
22	GRANT	SOUTHWEST	58	SHAWANO	NORTH CENTRAL
23	GREEN	SOUTHWEST	59	SHEBOYGAN	NORTHEAST
24	GREEN LAKE	NORTH CENTRAL	60	TAYLOR	NORTHWEST
25	IOWA	SOUTHWEST	61	TREMPEALEAU	NORTHWEST
26	IRON	NORTH CENTRAL	62	VERNON	SOUTHWEST
27	JACKSON	NORTHWEST	63	VILAS	NORTH CENTRAL
28	JEFFERSON	SOUTHWEST	64	WALWORTH	SOUTHEAST
29	JUNEAU	SOUTHWEST	65	WASHBURN	NORTHWEST
30	KENOSHA	SOUTHEAST	66	WASHINGTON	SOUTHEAST
31	KEWAUNEE	NORTHEAST	67	WAUKESHA	SOUTHEAST
32	LA CROSSE	SOUTHWEST	68	WAUPACA	NORTH CENTRAL
33	LAFAYETTE	SOUTHWEST	69	WAUSHARA	NORTH CENTRAL
34	LANGLADE	NORTH CENTRAL	70	WINNEBAGO	NORTHEAST
35	LINCOLN	NORTH CENTRAL	71	WOOD	NORTH CENTRAL
36	MANITOWOC	NORTHEAST	73	MENOMINEE	NORTH CENTRAL

Figure 2.1-3
Region Map

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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.

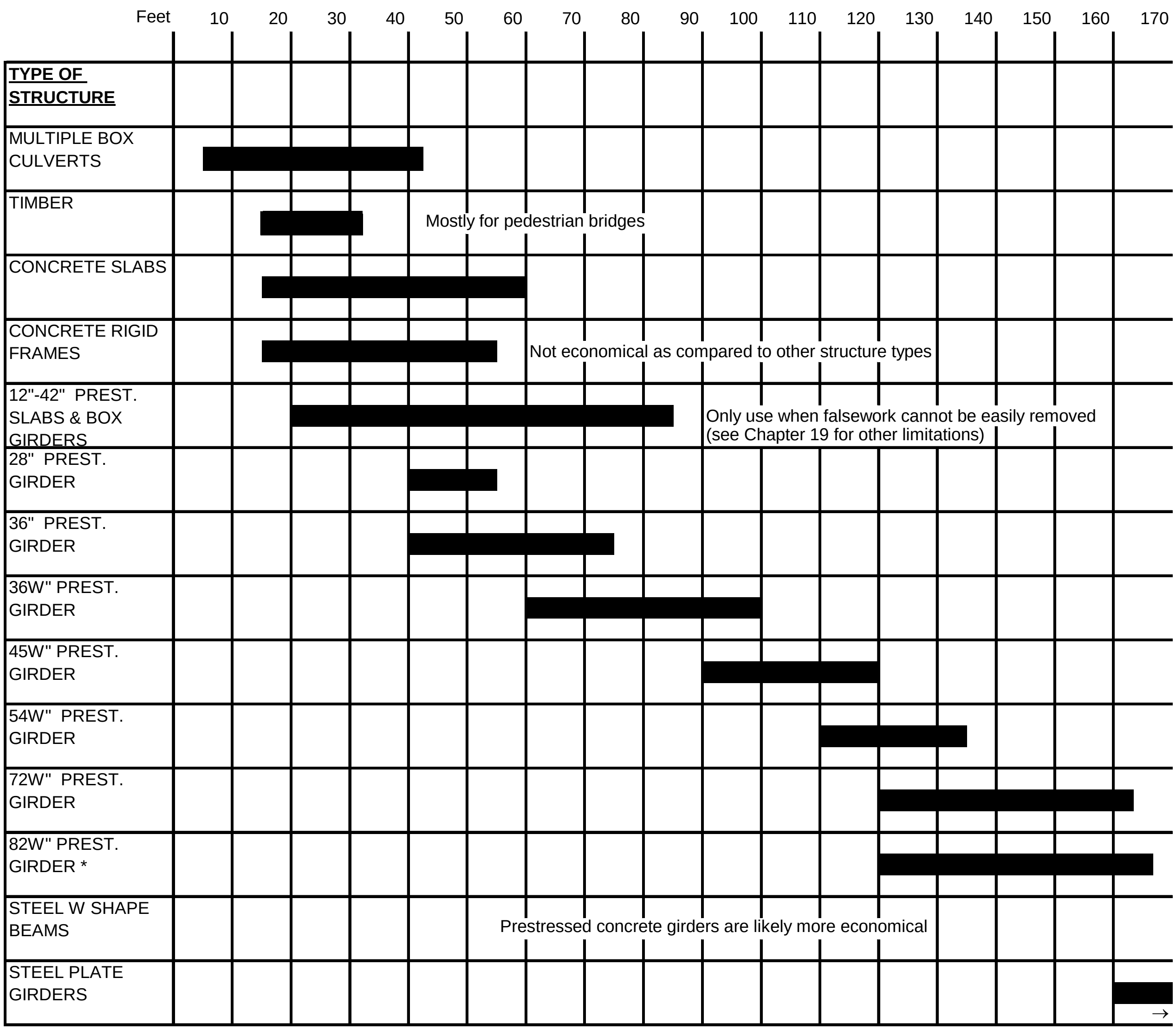
In these reports prestressed girder costs are grouped together because there is a small cost difference between girder sizes. Refer to unit costs. Concrete slab costs are also grouped together for this reason.

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.

Note: Current costs are given in English units.

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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Discharge	DISCH.	Near Side, Far Side	N.S.F.S.
Per Cent	%	Sidewalk	SDWK.
Plate	PL	South	S.
Point of Curvature	P.C.	Space	SPA.
Point of Intersection	P.I.	Specification	SPEC
Point of Tangency	P.T.	Standard	STD.
Point on Curvature	P.O.C.	Station	STA.
Point on Tangent	P.O.T.	Structural	STR.
Property Line	P.L.	Substructure	SUBST.
Quantity	QUAN.	Superstructure	SUPER.
Radius	R.	Surface	SURF.
Railroad	R.R.	Superelevation	S.E.
Railway	RY.	Symmetrical	SYM
Reference	REF.	Tangent Line	TAN. LN.
Reinforcement	REINF.	Transit Line	T/L
Reinforced Concrete Culvert Pipe	R.C.C.P.	Transverse	TRAN.
Required	REQ'D.	Variable	VAR.
Right	RT.	Vertical	VERT.
Right Hand Forward	R.H.F.	Vertical Curve	V.C.
Right of Way	R/W	Volume	VOL.
Roadway	RDWY.	West	W.
Round	ø	Zinc Gauge	ZN. GA.
Section	SEC.		

Table 6.3-1
Abbreviations

6.3.1.8 Nomenclature and Definitions

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

6.3.2 Plan Sheets

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

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

1. General Plan

2. Subsurface Exploration

3. Abutments

4. Piers

5. Superstructure and Superstructure Details

6. Railing and Parapet Details

Show all views looking up station.

6.3.2.1 General Plan (Sheet 1)

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

1. Plan View

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

   a. Sufficient dimensions to layout structure in the field.

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

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

   On Structure Replacements

   Show existing structure in dashed-lines on Plan View.

2. Elevation View

   Same requirements as specified for preliminary plan except:

   a. Show elevation at bottom of all substructure units.

   b. Give estimated pile lengths where used.

3. Cross-Section View

   Same requirements as specified for preliminary plan except:

11. Inspection Reports - New Bridge File - The Structures Maintenance Section loads a copy of the following Inspection Reports into the New Bridge File.

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

Table 6.3-2
Various Inspection Reports

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

6.3.5 Processing Plans

1. Before P.S. & E. Process

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

   Maintain plans as PDF on E-plan server.

2. At P.S. & E. Processing

   Prepare plans for bid letting process.

3. After Structure Construction

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

   Original plan sheets and Design Folders are discarded.

6.4 Computation of Quantities

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

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

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

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

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

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

6.4.1 Excavation for Structures Bridges (Structure)

This is a lump sum bid item. The limits of excavation are shown in the chapter in the manual which pertains to the structural item, abutments, piers, retaining walls, box culverts, etc.

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

6.4.2 Backfill Granular or Backfill Structure

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

6.4.3 Concrete Masonry Bridges

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

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13.1 General

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

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

13.1.1 Pier Type and Configuration

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

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

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

13.1.2 Bottom of Footing Elevation

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

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

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

13.1.3 Pier Construction

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

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

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

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

T = Temperature change of superstructure (°F)

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

h = Column height (ft)

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

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

The temperature force on a single fixed pier in a bridge is the resultant of the unbalanced forces acting on the substructure units. Maximum friction coefficients are assumed for expansion bearings on one side of the pier and minimum coefficients are assumed on the other side to produce the greatest unbalanced force on the fixed pier.

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

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

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

Where:

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

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

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

The values for computing temperature forces in Table 13.4-2 shall be used on Wisconsin bridges. Do not confuse this temperature change with the temperature range used for expansion joint design.

	Reinforced Concrete	Steel
Temperature Change	45 °F	90 °F
Coefficient of Thermal Expansion	0.0000060/°F	0.0000065/°F

Table 13.4-2
Temperature Expansion Values

Temperature forces on bridges with two or more fixed piers are based on the movement of the superstructure along its centerline. These forces are assumed to act normal and parallel to the longitudinal axis of the pier as resolved through the skew angle. The lateral restraint offered by the superstructure is usually ignored. Except in unusual cases, the larger stiffness generated by considering the transverse stiffness of skewed piers is ignored.

13.4.6 Force of Stream Current

The force of flowing water, WA, acting on piers is specified in **LRFD [3.7.3]**. This force acts in both the longitudinal and transverse directions.

13.4.6.1 Longitudinal Force

The longitudinal force is computed as follows:

$$p = \frac{C_D V^2}{1{,}000}$$

Where:

p	=	Pressure of flowing water (ksf)
V	=	Water design velocity for the design flood in strength and service limit states and for the check flood in the extreme event limit state (ft/sec)
C_D	=	Drag coefficient for piers (dimensionless), equal to 0.7 for semicircular-nosed piers, 1.4 for square-ended piers, 1.4 for debris lodged against the pier and 0.8 for wedged-nosed piers with nose angle of 90° or less

The longitudinal drag force shall be computed as the product of the longitudinal stream pressure and the projected exposed pier area.

13.4.6.2 Lateral Force

The lateral force is computed as follows:

$$p = \frac{C_D V^2}{1{,}000}$$

Where:

p	=	Lateral pressure of flowing water (ksf)

WisDOT policy item:

Since the angle of inclination of the pier nose with respect to the vertical is always less than or equal to 15° on standard piers in Wisconsin, the flexural ice failure mode does not need to be considered for these standard piers ($f_b = 0$).

WisDOT policy item:

If the pier is approximately aligned with the direction of the ice flow, only the first design case as specified in **LRFD [3.9.2.4]** shall be investigated due to the unknowns associated with the friction angle defined in the second design case.

A longitudinal force equal to F shall be combined with a transverse force of 0.15F

Both the longitudinal and transverse forces act simultaneously at the pier nose.

If the pier is located such that its longitudinal axis is skewed to the direction of the ice flow, the ice force on the pier shall be applied to the projected pier width and resolved into components. In this condition, the transverse force to the longitudinal axis of the pier shall be a minimum of 20% of the total force.

WisDOT exception to AASHTO:

Based upon the pier geometry in the Standards, the ice loadings of **LRFD [3.9.4]** and **LRFD [3.9.5]** shall be ignored.

13.4.8.2 Force Exerted by Expanding Ice Sheet

Expansion of an ice sheet, resulting from a temperature rise after a cold wave, can develop considerable force against abutting structures. This force can result if the sheet is restrained between two adjacent bridge piers or between a bluff type shore and bridge pier. The force direction is therefore transverse to the direction of stream flow.

Force from ice sheets depends upon ice thickness, maximum rate of air-temperature rise, extent of restraint of ice and extent of exposure to solar radiation. In the absence of more precise information, estimate an ice thickness and use a force of 8.0 ksf.

It is not necessary to design all bridge piers for expanding ice. If one side of a pier is exposed to sunlight and the other side is in the shade along with the shore in the pier vicinity, consider the development of pressure from expanding ice. If the central part of the ice is exposed to the sun's radiation, consider the effect of solar energy, which causes the ice to expand.

13.4.9 Centrifugal Force

Centrifugal force, CE, is specified in **LRFD [3.6.3]** and is included in the pier design for structures on horizontal curves. The lane load portion of the HL-93 loading is neglected in the computation of the centrifugal force.

The centrifugal force is taken as the product of the axle weights of the design truck or tandem and the factor, C, given by the following equation:

$$C = \frac{4}{3}\frac{v^2}{gR}$$

Where:

V	=	Highway design speed (ft/sec)
g	=	Gravitational acceleration = 32.2 (ft/sec^2)
R	=	Radius of curvature of travel lane (ft)

The multiple presence factors specified in **LRFD [3.6.1.1.2]** shall apply to centrifugal force.

Centrifugal force is assumed to act radially and horizontally 6’ above the roadway surface. The point 6’ above the roadway surface is measured from the centerline of roadway. The design speed may be determined from the Wisconsin *Facilities Development Manual*, Chapter 11. It is not necessary to consider the effect of superelevation when centrifugal force is used, because the centrifugal force application point considers superelevation.

13.4.10 Extreme Event Collision Loads

WisDOT policy item:

With regards to **LRFD [3.6.5]** and vehicular collision force, CT, protecting the pier and designing the pier for the 400 kip static force are each equally acceptable. The bridge design engineer should work with the roadway engineer to determine which alternative is preferred.

WisDOT policy item:

Designs for bridge piers adjacent to roadways with a design speed ≤ 40 mph need not consider the provisions of **LRFD [3.6.5]**. As for all multi-columned piers, a minimum of 3 columns is still required.

If the design speed of a roadway adjacent to a pier is > 40 mph and the pier is not protected by a TL-5 barrier, embankment or adequate offset, the pier columns/shaft, *only*, shall be strengthened to comply with **LRFD [3.6.5]**. For a multi-column pier the minimum size column shall be 3x5 ft rectangular or 4 ft diameter (consider clearance issues and/or the wide cap required when using 4 ft diameter columns). Solid shaft and hammerhead pier shafts are considered adequately sized.

The vertical reinforcement for the columns/shaft shall be the greater of what is required by design (not including the Extreme Event II loading) or a minimum of 1.5% of the gross concrete section (total cross section without deduction for rustications less than or equal to 1-1/2" deep) to address the collision force for the 3x5 ft rectangular and 4 ft diameter columns. The 1.5% minimum for 15 sq. ft. may be prorated down to 1% minimum for sections with at least a 30 sq. ft. cross sectional area.

For the 3x5 ft rectangular columns, use double #5 stirrups spaced at 6" vertically as a minimum. For the 4 ft diameter columns, use #4 spiral reinforcement (smooth bars) spaced vertically at 6" as a minimum. Hammerhead pier shafts shall have, as a minimum, the horizontal reinforcement as shown on the Standards.

WisDOT exception to AASHTO:

The vessel collision load, CV, in **LRFD [3.14]** will not be applied to every navigable waterway of depths greater than 2'. For piers located in navigable waterways, the engineer shall contact the WisDOT project manager to determine if a vessel collision load is applicable.

13.5 Load Application

When determining pier design forces, a thorough understanding of the load paths for each load is critical to arriving at loads that are reasonable per *AASHTO LRFD*. The assumptions associated with different pier, bearing and superstructure configurations are also important to understand. This section provides general guidelines for the application of forces to typical highway bridge piers.

13.5.1 Loading Combinations

Piers are designed for the Strength I, Strength III, Strength V and Extreme Event II load combinations as specified in **LRFD [3.4.1]**. Reinforced concrete pier components are also checked for the Service I load combination. Load factors for these load combinations are presented in Table 13.5-1. See 13.10 for loads applicable to pile bents and pile encased piers.

Load Combination Limit State	Load Factor										
	DC		DW		LL+IM BR CE	WA	WS	WL	FR	TU CR SH	IC CT CV
	Max.	Min.	Max.	Min.							
Strength I	1.25	0.90	1.50	0.65	1.75	1.00	0.00	0.00	1.00	0.50	0.00
Strength III	1.25	0.90	1.50	0.65	0.00	1.00	1.40	0.00	1.00	0.50	0.00
Strength V	1.25	0.90	1.50	0.65	1.35	1.00	0.40	1.00	1.00	0.50	0.00
Service I	1.00	1.00	1.00	1.00	1.00	1.00	0.30	1.00	1.00	1.00	0.00
Extreme Event II	1.25	0.90	1.50	0.65	0.50	1.00	0.00	0.00	1.00	0.00	1.00

Table 13.5-1
Load Factors

13.5.2 Expansion Piers

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

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

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

13.9 Column / Shaft Design

See 13.4.10 for minimum shaft design requirements regarding the Extreme Event II collision load of **LRFD [3.6.5]**.

Use an accepted analysis procedure to determine the axial load as well as the longitudinal and transverse moments acting on the column. These forces are generally largest at the top and bottom of the column. Apply the load factors for each applicable limit state. The load factors should correspond to the gross moment of inertia. Load factors vary for the gross moment of inertia versus the cracked moment as defined in **LRFD [3.4.1]** for γ_{TU}, γ_{CR}, γ_{SH}. Choose the controlling load combinations for the column design.

Columns that are part of a pier frame have transverse moments induced by frame action from vertical loads, wind loads on the superstructure and substructure, wind loads on live load, thermal forces and centrifugal forces. If applicable, the load combination for Extreme Event II must be considered. Longitudinal moments are produced by the above forces, as well as the braking force. These forces are resolved through the skew angle of the pier to act transversely and longitudinally to the pier frame. Longitudinal forces are divided equally among the columns.

Wisconsin uses tied columns following the procedures of **LRFD [5.7.4]**. The minimum allowable column size is 2'-6" in diameter. The minimum steel bar area is as specified in **LRFD [5.7.4.2]**.

The computed column moments are to consider moment magnification factors for slenderness effects as specified in **LRFD [5.7.4.3]**. Values for the effective length factor, K, are as follows:

- 1.2 for longitudinal moments with a fixed seat supporting prestressed concrete girders
- 2.1 for longitudinal moments with a fixed seat supporting steel girders and all expansion bearings
- 1.0 for all transverse moments

The computed moments are multiplied by the moment magnification factors, if applicable, and the column is designed for the combined effects of axial load and bending. According to **LRFD [5.7.4.1]** all force effects, including magnified moments, shall be transferred to adjacent components. The design resistance under combined axial load and bending is based on stress-strain compatibility. A computer program is recommended for determining the column's resistance to the limit state loads.

As a minimum, the column shall provide the steel shown on the Pier Standards.

On large river crossings, it may be necessary to protect the piers from damage. The noses may be protected by using steel with anchors embedded into the concrete. Dolphins may also be provided.

The column-to-cap connection is designed as a rigid joint considering axial and bending stresses. Column steel is run through the joint into the cap to develop the compressive stresses or the tensile steel stresses at the joint.

In general, the column-to-footing connection is also designed as a rigid joint. The bar steel from the column is generally terminated at the top of the footing. Dowel bars placed in the footing are used to transfer the steel stress between the footing and the column.

13.9.1 Tapered Columns of Concrete and Timber

Design tapered columns of concrete and timber using the existing column formulas, taking the cross-sectional area at the small end. However, d, as used in L/d, is taken as follows:

1. For round columns or rectangular columns tapered in both directions, use:

$$d = d_B$$

2. For rectangular columns tapered in the plane of bending only, use:

$$d = (d_A)2(d_B)8$$

3. For rectangular columns tapered perpendicular to the plane of bending, use:

$$d = (d_A)7(d_B)3$$

Where:

d_A = Dimension at the small end

d_B = Dimension at the large end

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14.1 Introduction

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

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

14.1.1 Wall Development Process

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

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

Based on the SSR, a Geotechnical site investigation may be required to determine foundation and retained soil properties. A hydraulic analysis is also conducted, if required, to asses scour potential. The Geotechnical investigation generally includes a subsurface and laboratory investigation. For the departmental-designed walls, the Bureau of Technical Services, Foundation & Pavement Unit (Geotechnical Unit) can recommend the scope of soil exploration needed and provide/recommend bearing resistance, overall stability, and settlement of walls based on the geotechnical exploration results.

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

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

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

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

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

For further information related to wall selection, design, approval process, pre-approval and review of proprietary wall systems please contact Structures Design Section of the Bureau of Structures at 608-266-8489. For questions pertaining to geotechnical analyses and geotechnical investigations please contact the Geotechnical Unit at 608-246-7940.

14.1.1.1 Wall Numbering System

Permanent retaining walls that are designed for a design life of 75 years or more should be identified by a wall number, R-XX-XXX, as assigned by the Region. For a continuous wall consisting of various wall types, the numbering system should include unit numbers so that the numbering appears as R-XX-XXX-001, R-XX-XXX-002, and so on. The first two digits represent the county the wall is located in and the next set(s) of digits represent the undivided wall.

The wall number should be assigned in accordance with 2.5 of this manual. The only walls not requiring a number are cast-in-place concrete walls being utilized as bridge abutment wings and those walls whose height does not exceed 5.0 foot at any given point along the wall length. Wall height is measured from top of leveling pad or footing to the bottom of wall cap.

14.2 Wall Types

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

Conventional Walls

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

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

Permanent or Temporary Walls

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

Fill Walls or Cut Walls

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

Bottom-up or Top-down Constructed Walls

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

Proprietary or Non-Proprietary

Some retaining walls have prefabricated modules or components that are proprietary in nature. Based on the use of proprietary components, walls can be divided into the categories of proprietary and non-proprietary wall systems as defined in 14.1.1.

A proprietary retaining wall system is considered as a patented or trademarked retaining wall system or a wall system comprised of elements/components that are protected by a trade name, brand name, or patent and are designed and supported by the manufacturer. MSE walls, modular block gravity walls, bin, and crib walls are considered proprietary walls because these walls have components which are either patented or have trademarks.

Proprietary walls require preapproval and appropriate special provisions. The preapproval requirements are discussed in 14.16 of this chapter. Proprietary walls also have special design requirements for the structural components, and are discussed in further detail within each specific wall design section. Most MSE, modular block, bin or crib walls require pre-approval and/or special provisions.

A non-proprietary retaining wall is fully designed and detailed by the designer or may be design-build. A non-proprietary retaining wall system may contain proprietary elements or components as well as non-proprietary elements and components. CIP cantilever walls, rock walls, soil nail walls and non-gravity walls fall under this category.

Wall classification is shown in Table 14.2-1 and is based on wall type, project function category, and method of construction.

14.2.1 Gravity Walls

Gravity walls are considered externally stabilized walls as these walls use self weight to resist lateral pressures due to earth and water. Gravity walls are generally subdivided into mass gravity, semi-gravity, modular gravity, mechanically stabilized reinforced earth (MSE), and in-situ reinforced earth wall (soil nailing) categories. A schematic diagram of the various types of gravity walls is included in Figure 14.2-1.

14.2.1.1 Mass Gravity Walls

A mass gravity wall is an externally stabilized, cast-in-place rigid gravity wall, generally trapezoidal in shape. The construction of these walls requires a large quantity of materials so these are rarely used except for low height walls less than 8.0 feet. These walls mainly rely on self weight to resist external pressures and their construction is staged as bottom up construction, mostly in fill or cut/fill situation.

14.2.1.2 Semi-Gravity Walls

Semi-gravity walls resist external forces by the combined action of self weight, weight of soil above footing and the flexural resistance of the wall components. A cast-in-place (CIP) concrete cantilever wall is an example and consists of a reinforced concrete stem and a base footing. These walls are non-proprietary.

Cantilever walls are best suited for use in areas exhibiting good bearing material. When bearing or settlement is a problem, these walls can be founded on piles or foundation improvement may be necessary. Walls exceeding 28 feet in height are provided with counter-forts or buttress slabs. Construction of these walls is staged as bottom-up construction and mostly constructed in fill situations. Cantilever walls are more suited where MSE walls are not feasible, although these walls are generally costlier than MSE walls.

14.2.1.3 Modular Gravity Walls

Modular walls are also known as externally stabilized gravity walls as these walls resist external forces by utilizing self weight. Modular walls have prefabricated modules/components which are considered proprietary. The construction is bottom-up construction mostly used in fill situations.

14.2.1.3.1 Modular Block Gravity Walls

Modular block concrete facings are used without soil reinforcement to function as an externally stabilized gravity wall. The modular blocks are prefabricated dry cast or wet cast concrete blocks and the blocks are stacked vertically or slightly battered to resist external forces. The concrete blocks are either solid concrete or hollow core concrete blocks. The hollow core concrete blocks are filled with crushed aggregates or sand. The walls are limited to a maximum design height of 8 feet. The modular blocks are proprietary and vary in sizes.

14.2.1.3.2 Prefabricated Bin, Crib and Gabion Walls

<u>Bin Walls</u>: Concrete and metal bin walls are built of adjoining open or closed faced bins and then filled with soil/rocks. Each metal bin is comprised of individual members bolted. The concrete bin wall is comprised of prefabricated interlocking concrete modules. These wall systems are proprietary wall systems.

<u>Crib Walls</u>: Crib walls are constructed of interlocking prefabricated units of reinforced or unreinforced concrete or timber elements. Each crib is comprised of longitudinal and transverse members. Each unit is filled with free draining material. These wall systems are proprietary wall systems.

<u>Gabion Walls</u>**:** Gabion walls are constructed of steel wire baskets filled with selected rock fragments and tied together. Gabions walls are flexible, free draining and easy to construct. These wall systems are proprietary wall systems. Maximum heights are normally less than 21 feet. These walls are desirable where equipment access is limited. The wires used for constructing gabions baskets must be designed with adequate corrosion protection.

14.2.1.4 Rock Walls

Rock walls are also known as 'Rockery Walls'. These types of gravity walls are built by stacking locally available large stones or boulders into a trapezoid shape. These walls are highly flexible and height of these walls is generally limited to approximately 8.0 feet. A layer of gravel and geotextile is commonly used between the stones and the retained soil. These walls can be designed using the *FHWA Rockery Design and Construction Guideline*.

14.2.1.5 Mechanically Stabilized Earth (MSE) Walls:

Mechanically Stabilized Earth (MSE) walls include a selected soil mass reinforced with metallic or geo-synthetic reinforcement. The soil reinforcement is connected to a facing element to prevent the reinforced soil from sloughing. Construction of these walls is staged as bottom-up construction. These can be constructed in cut and fill situations, but are better suited to fill sites. MSE walls are normally used for wall heights between 10 to 40 feet. A brief description of various types of MSE walls is given below:

Precast Concrete Panel MSE Walls: These types of walls employ a metallic strip or wire grid reinforcement connected to precast concrete panels to reinforce a selected soil mass. The concrete panels are usually 5'x5' or 5'x10' size panels. These walls are proprietary wall systems.

Modular Block Facing MSE Wall: Prefabricated modular concrete block walls consist of almost vertically stacked concrete modular blocks and the soil reinforcement is secured between the blocks at predetermined levels. Metallic strips or geogrids are generally used as soil reinforcement to reinforce the selected soil mass. Concrete blocks are either solid or hollow core blocks. The hollow core blocks are filled with aggregates or sand. These types of walls are proprietary wall systems.

Geotextile/Geogrids/Welded Wire Faced MSE Walls: These types of MSE walls consist of compacted soil layers reinforced with continuous or semi-continuous geotextile, geogrid or welded wire around the overlying reinforcement. The wall facing is formed by wrapping each layer of reinforcement around the overlying layer of backfill and re-embedding the free end into the backfill. These types of walls are used for temporary or permanent applications. Permanent facings include shotcrete, gunite, galvanized welded wire mesh, cast-in-place concrete or prefabricated concrete panels.

14.2.1.6 Soil Nail Walls

Soil nail walls are internally stabilized cut walls that use in-situ reinforcement for resisting earth pressures. The large diameter rebars (generally #10 or greater) are typically used for the reinforcement. The construction of soil nail walls is staged top-down and soil nails are installed after each stage of excavation. Shotcrete can be applied as a facing. The facing of a soil nail wall is typically covered with vertical drainage strips located over the nail then covered with shotcrete. Soil nailing walls are used for temporary or permanent construction. Specialty contractors are required when constructing these walls. Soil nail walls have been installed to heights of 60.0 feet or more but there have only been a few soil nail walls constructed on WisDOT projects.

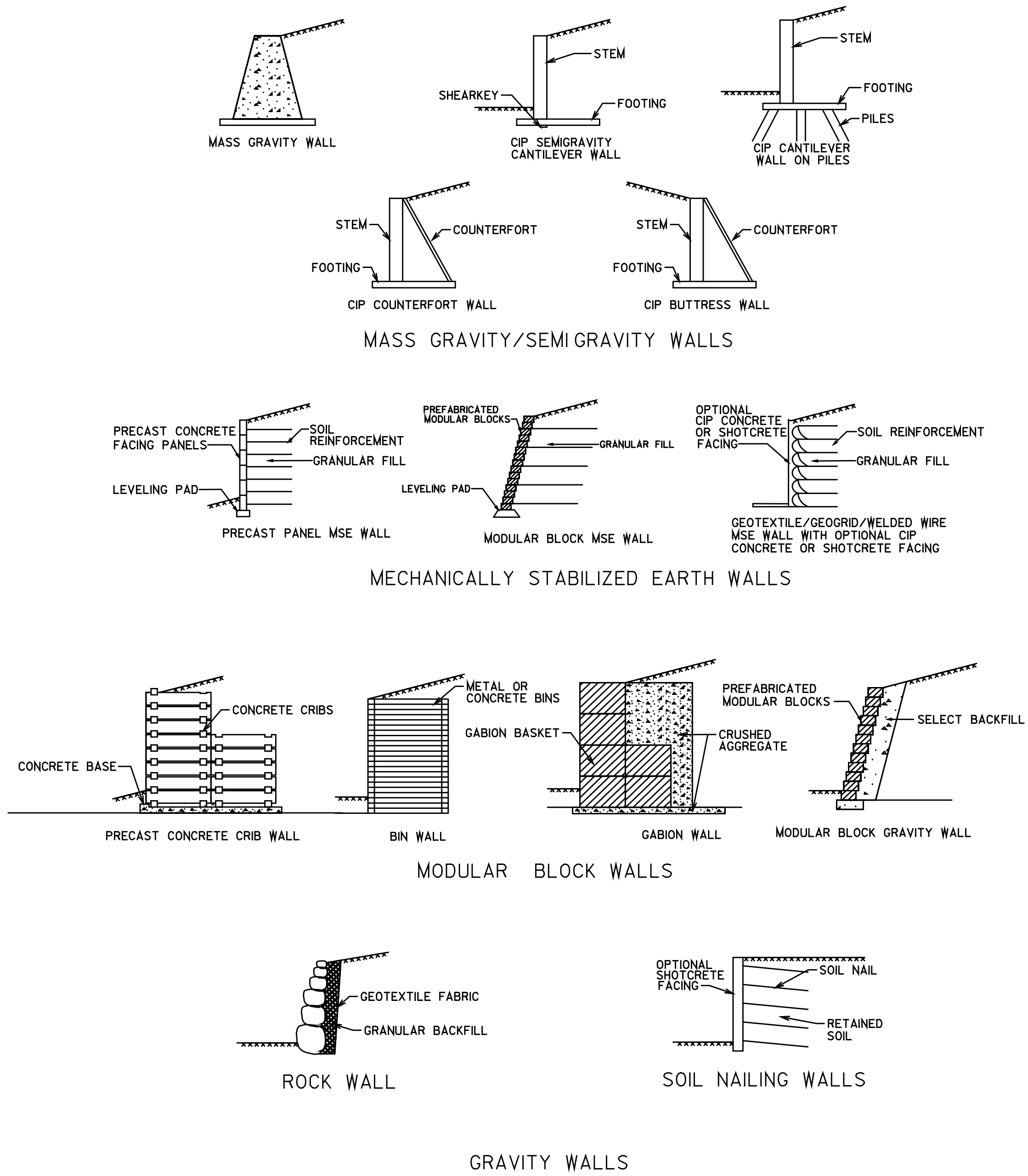

Figure 14.2-1
Gravity Walls

14.2.2 Non-Gravity Walls

Non-gravity walls are classified into cantilever and anchored wall categories. These walls are considered as externally stabilized walls and used in cut situations. The walls include sheet pile, post and panel, tangent and secant pile type with or without anchors. Figure 14.2-2 shows common types of non-gravity walls.

14.2.2.1 Cantilever Walls

These types of walls derive lateral resistance through embedment of vertical elements into natural ground and the flexure resistance of the structural members. They are used where excavation support is needed in shallow cut situations.

Cantilever Sheet Pile Walls: Cantilever sheet pile walls consist of interlocking steel panels, driven into the ground to form a continuous sheet pile wall. The sheet piles resist the lateral earth pressure utilizing the passive resistance in front of the wall and the flexural resistance of the sheet pile. Most sheet pile walls are less than 15 feet in height.

Soldier Pile Walls: These types of walls are non gravity wall systems that derive lateral resistance and moment capacity through embedment of vertical members (soldier piles) into natural ground in cut situations. The vertical elements may be drilled or driven steel or concrete members. The soil behind the wall is retained by lagging. The lagging may be steel, wood, or concrete.

Post and Panel Walls: These types of walls are comprised of vertical elements (usually H piles) and concrete panels which extend between vertical elements. The panels are usually constructed of precast reinforced concrete or precast prestressed concrete. These walls should be considered when disturbance to the site is critical. These are also suitable for site where rock is encountered near surface. Post and panel walls are constructed from bottom up.

Tangent and Secant Pile Walls: A tangent pile wall consists of a single row of reinforced concrete piles (drilled) installed in the ground. Each pile touches the adjacent pile tangentially. The concrete piles are reinforced using a single steel beam or a cage of reinforcing bars. A secant wall, generally, consists of a single row of overlapping and alternating reinforced and unreinforced piles drilled into the ground. Secant and tangent wall systems are used to hold earth and water where water tightness is important, and lowering of the water table is not desirable.

14.2.2.2 Anchored Walls

Anchored walls are externally stabilized non-gravity cut walls. Anchored walls are essentially the same as cantilever walls except that these walls utilize anchors (tiebacks) to extend the wall heights beyond the design limit of the cantilever walls. These walls require less toe embedment than cantilever walls.

These walls derive lateral resistance by embedment of vertical wall elements into firm ground and by anchorages. Most commonly used anchored walls are anchored sheet pile walls and the soldier pile walls. Tangent and secant walls can also be anchored with tie backs and used as anchored walls. The anchors can be attached to the walls by tie rods, bars or wired tendons. The anchoring device is generally a deadman, screw-type, or grouted tieback anchor. Anchored walls can be built to significant heights.

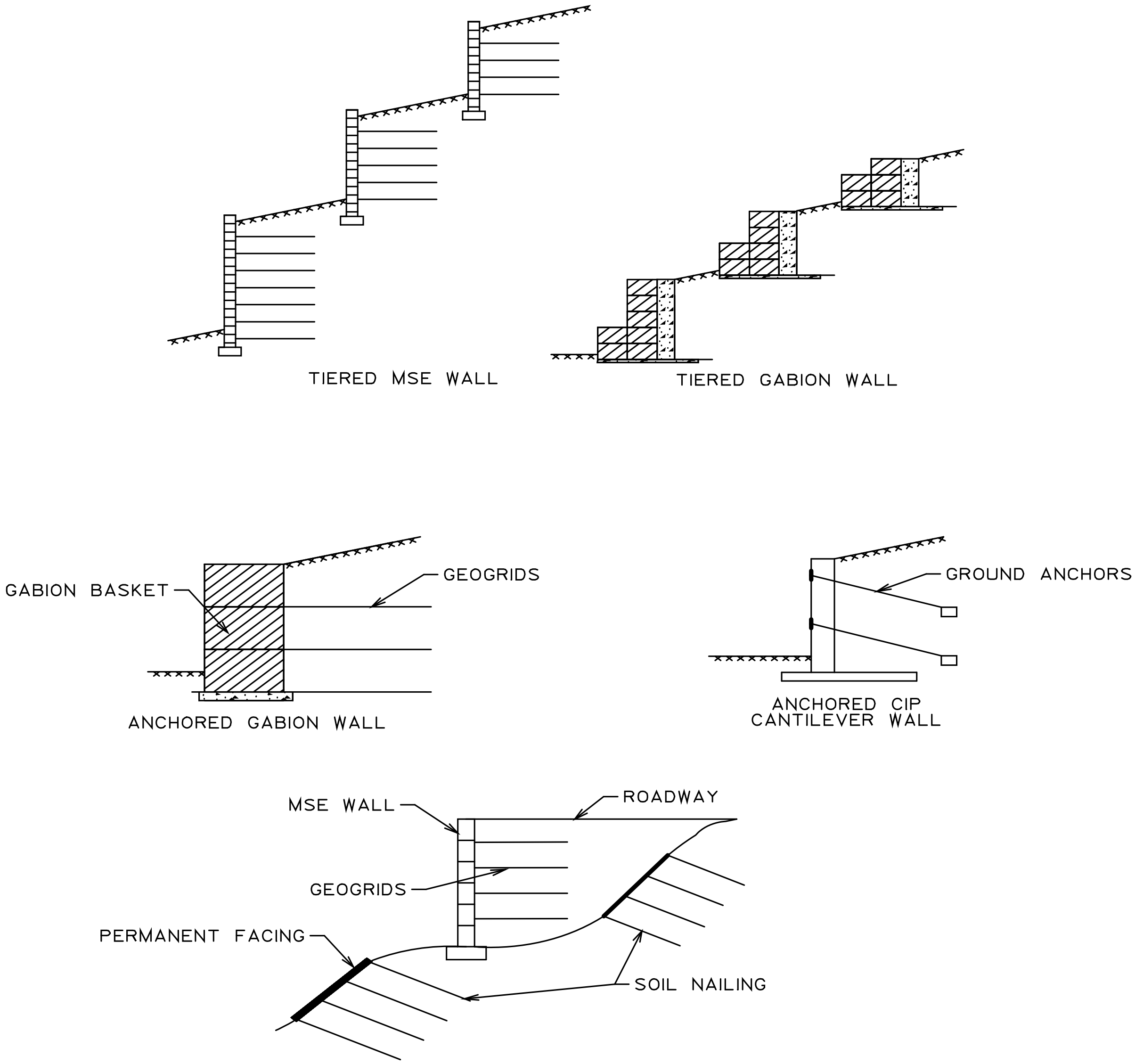

Figure 14.2-3
Tiered & Hybrid Wall Systems

Wall Category	Wall Sub-Category	Wall Type	Typical Construction Concept	Proprietary
Gravity	Mass Gravity	CIP Gravity	Bottom Up (Fill)	No
	Semi-Gravity	CIP Cantilever	Bottom Up (Fill)	No
	Reinforced Earth	MSE Walls- Precast Panel, Modular Blocks, Geogrid/ Geo-textile/Wire-Faced	Bottom Up (Fill)	Yes
	Modular Gravity	Modular Blocks, Gabion, Bin, Crib	Bottom Up/(Fill)	Yes
	In-situ Reinforced	Soil Nailing	Top Down (Cut)	No
Non-Gravity	Cantilever	Sheet Pile, Soldier Pile, Tangent/Secant	Top Down (Cut)	No
	Cantilever	Post and Panel, Tangent/Secant	Bottom up(Fill)	No
	Anchored	Anchored Sheet Pile, Soldier Pile, Tangent/Secant	Top Down (Cut)	No

Table 14.2-1
Wall Classification

improvements or utility construction should be allowed in the ROW area of the retaining wall systems.

14.3.1.6 Utilities and Other Conflicts

Feasibility of some wall systems may be influenced by the presence of utilities and buried structures. MSE, soil nailing and anchored walls commonly have conflict with the presence of utilities or buried underground structures. MSE walls should not be used where utilities must stay in the reinforcement zone.

14.3.1.7 Aesthetics

In addition to being functional and economical, the walls should be aesthetically pleasing. Wall aesthetics may influence selection of a particular wall system. However, the aesthetic treatment should complement the retaining wall and not disrupt the functionality or selection of wall type. All permanent walls should be designed with due considerations to the wall aesthetics. Each wall site must be investigated individually for aesthetic needs. Temporary walls should generally be designed with little consideration to aesthetics. Chapter 4 - Aesthetics presents structures aesthetic requirements.

14.3.1.8 Constructability Considerations

Availability of construction material, site accessibility, equipment availability, form work and temporary shoring, dewatering requirements, labor considerations, complicated alignment changes, scheduling consideration, speed of construction, construction staging/phasing and maintaining traffic during construction are some of the important key factors when evaluating the constructability of each wall system for a specific site project.

In addition, it should also be ensured that the temporary excavation slopes used for wall construction are stable as per site conditions and meet all safety requirements laid by Occupation and Safety Health Administration (OSHA).

14.3.1.9 Environmental Considerations

Selection of a retaining wall system is influenced by its potential environmental impact during and after construction. Some of the environmental concerns during construction may include excavation and disposal of contaminated material at the project site, large quantity of water, corrosive nature of water, vibration impacts, noise abatement and pile driving constraints.

14.3.1.10 Cost

Cost of a retaining wall system is influenced by many factors that must be considered while estimating preliminary costs. The components that influence cost include excavation, structure, procurement of additional easement or ROW, drainage, disposal of unsuitable material, traffic maintenance etc. Maintenance cost also affects overall cost of a retaining wall system. The retaining walls that have least structural cost may not be the most economical walls. Wall selection should be based on overall cost. When feasible, MSE Walls and modular block gravity walls cost less than other walls

14.3.1.11 Mandates by Other Agencies

In certain project locations, other agency mandates may limit the types of wall systems considered.

14.3.1.12 Requests made by the Public

A Public Interest Finding could dictate the wall system to be used on a specific project.

14.3.1.13 Railing

For safety reasons most walls will require a protective railing. The railing will usually be located behind the wall. The roadway designer will generally determine whether a pedestrian or non-pedestrian railing is required and what aesthetic considerations are needed.

14.3.1.14 Traffic barrier

A traffic barrier should be installed if vehicles, bicycles, or pedestrians are likely to be present on top of the wall. The roadway designer generally determines the need for a traffic barrier.

14.3.2 Wall Selection Guide Charts

Table 14.3-1 and Table 14.3-2 summarize the characteristics for the various wall types that are normally considered during the wall selection process. The tables also present some of the advantages, disadvantages, cost effective height range and other key selection factors. A wall designer can use these tables and the general wall selection criteria discussed in 14.3.1 as a guide. Designers are encouraged to contact the Structures Design Section if they have any questions relating to wall selection for their project.

Wall Type	Temp	Perm	Cost Effective Height (ft)	Req'd. ROW	Advantages	Disadvantages
Concrete Gravity		√	3-10	.5H-.7H	• Durable • Meets aesthetic requirement • Requires small quantity of select backfill	• High cost • May need deep foundation • Longer const. time
Reinforced CIP Cantilever		√	6-28	.4H-.7H	• Durable meets aesthetic requirement • Requires small quantity of select backfill	• High cost • May need deep foundation • Longer const. time & deeper embedment
Reinforced CIP Counterfort		√	26 -40	0.4H-0.7H	• Durable • Meets aesthetic requirement • Requires small back fill quantity	• High cost • May need deep foundation • Longer const. time & deeper embedment
Concrete Modular Block		√	3-8	.4H-.7H	• Does not require skilled labor or specialized equipment	• Height limitations
Metal Bin		√	6 -20	.4H-.7H	• Does not require skilled labor or special equipment	• Difficult to make height adjustment in the field
Concrete Crib		√	6-20	.4H-.7H	• Does not require skilled labor or specialized equipment	• Difficult to make height adjustment in the field
Gabion		√	6-20	.4H-.7H	• Does not require skilled labor or specialized equipment	• Need large stone quantities • Significant labor
MSE Wall (precast concrete panel with steel reinforcement)		√	10-35	.7H-1.0H	• Does not require skilled labor or specialized equipment	• Requires use of select backfill
MSE Wall (modular block and geo-synthetic reinforcement)		√	6-22	.7H-1.0H	• Does not require skilled labor or specialized equipment	• Requires use of select backfill
MSE Wall (geotextile/ geogrid / welded fire facing)	√	√	6-35	.7H-1.0H	• Does not require skilled labor or specialized equipment	• Requires use of select backfill

Table 14.3-1

Wall Selection Chart for Gravity Walls

Wall Type	Temp	Perm	Cost Effective Height (ft)	Req'd. ROW	Water Tightness	Advantages	Disadvantages
Sheet Pile	√	√	6-15	minimal	fair	• Rapid construction • Readily available	• Deep foundation may be needed • Longer construction time
Post & Panel	√	√	6-28	.2H-.5H	poor	• Easy construction • Readily available	• High cost • Deep foundation may be needed • Longer construction time
Tangent Pile		√	20 -60	.4H-.7H	good	• Adaptable to irregular layout • Can control wall stiffness	• High cost • Deep foundation may be needed • Longer construction
Secant Pile Wall		√	14-60	.4H-.7H	fair	• Adaptable to irregular layout • Can control wall stiffness	• Difficult to make height adjustment in the field • High cost
Anchored Wall	√	√	15-35	.4H-.7H	fair	• Rapid construction	• Difficult to make height adjustment in the field
Soil Nail Wall	√	√	6-20	.4H-.7H	fair	• Option for top-down	• Cannot be used in all soil types • Cannot be used below water table • Significant labor

Table 14.3-2
Wall Selection Chart for Non-Gravity Walls

14.6 Mechanically Stabilized Earth Retaining Walls

14.6.1 General Considerations

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

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

The MSE walls shall be designed for external stability of the wall system and internal stability of the reinforced soil mass. The overall stability shall also be considered as part of design evaluation. MSE walls are proprietary wall systems and the design responsibilities with respect to overall, external, and internal stability is shared between the designer (WisDOT or Consultant) and contractor. The designer is responsible for the overall or global, and preliminary external stability, whereas the contractor is responsible for the internal stability, final external stability and structural design of the wall. The responsibilities of the designer and contractor are outlined in 14.6.3.2. The design and drawings of MSE walls provided by the contractor must also be in compliance with the WisDOT special provisions as stated in 14.15.2 and 14.16

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

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

14.6.1.1 Usage Restrictions for MSE Walls

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

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

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

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

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

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

14.6.2 Structural Components

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

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

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

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17.1 Design Method

17.1.1 Design Requirements

All new structures and deck replacements are to be designed using *AASHTO LRFD Bridge Design Specifications*, hereafter referred to as *AASHTO LRFD*. Bridge rehabilitations and widenings are to be designed using either LFD or LRFD, at the designer's option.

LRFD utilizes load combinations called limit states which represent the various loading conditions which structural materials must be able to withstand. Limit states have been established in four major categories – strength, service, fatigue and extreme event. Different load combinations are used to analyze a structure for certain responses such as deflections, permanent deformations, ultimate strength and inelastic responses without failure. When all applicable limit states and combinations are satisfied, a structure is deemed acceptable under the LRFD design philosophy.

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

17.1.2 Rating Requirements

All new structures and deck replacements are rated for *AASHTO LRFD* (HL-93) live loads. Rating factors, RF, for inventory and operating rating are shown on the plans. Ratings will be based on *Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges*, hereafter referred to as *AASHTO LRFR*. See Chapter 45 – Bridge Rating for rating requirements.

17.1.2.1 Standard Permit Design Check

New structures are also to be checked for the Wisconsin Standard Permit Vehicle (Wis-SPV). The structure should have a minimum capacity to carry a gross vehicle load of 190 kips, while also supporting the future wearing surface. This truck will be designated as a Single Trip Permit Vehicle. It will have no escorts restricting the presence of other traffic on the bridge, no lane position restrictions imposed and no restrictions on speed to reduce the dynamic load allowance, IM.

The maximum Wisconsin Standard Permit Vehicle load that the structure can resist, calculated including current wearing surface loads, is shown on the plans.

See Chapter 45 – Bridge Rating for details about the Wisconsin Standard Permit Vehicle and calculating the maximum load for this permit vehicle.

17.2 LRFD Requirements

17.2.1 General

For superstructure member design, the component dimensions and the size and spacing of reinforcement shall be selected to satisfy the following equation for all appropriate limit states, as presented in **LRFD [1.3.2.1]**:

$$Q = \sum \eta_i \gamma_i Q_i \leq \phi R_n = R_r$$

Where:

η_i	=	Load modifier (a function of η_D, η_R, and η_i)
γ_i	=	Load factor
Q_i	=	Force effect: moment, shear, stress range or deformation caused by applied loads
Q	=	Total factored force effect
ϕ	=	Resistance factor
R_n	=	Nominal resistance: resistance of a component to force effects
R_r	=	Factored resistance = ϕR_n

17.2.2 WisDOT Policy Items

WisDOT policy items:

Set the value of the load modifier, η_i (see **LRFD [1.3.2.1]**), and its factors, η_D, η_R and η_I, all equal to 1.00.

Ignore any influence of ADTT on multiple presence factor, m, in **LRFD [Table 3.6.1.1.2-1]** that would reduce force effects.

17.2.3 Limit States

The following limit states (as defined in **LRFD [3.4.1]**) are utilized by WisDOT in the design of bridge superstructures.

span bridge in which the maximum negative moment at the pier is being computed and the second and third axles are positioned in different spans. The design truck is described in **LRFD [3.6.1.2.2]**.

17.2.4.2.2 Design Tandem

The design tandem has two axles, each with a loading of 25 kips and an axle spacing of 4 feet, as presented in Figure 17.2-2. The design tandem is described in **LRFD [3.6.1.2.3]**.

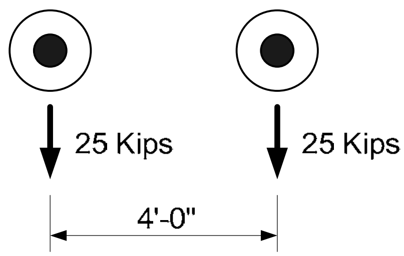


Figure 17.2-2
Design Tandem

WisDOT policy item:

WisDOT does not consider the use of dual tandems for negative moments and reactions, as suggested in **LRFD [C3.6.1.3.1]**. The design engineer shall receive direction from the owner and the BOS if this load is to be applied.

17.2.4.2.3 Design Lane

The design lane has a uniform load of 0.64 kips per linear foot, distributed in the longitudinal direction, as presented in Figure 17.2-3. The design lane is described in **LRFD [3.6.1.2.4]**.

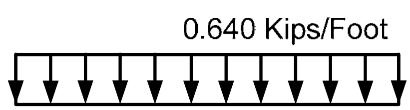


Figure 17.2-3
Design Lane

17.2.4.2.4 Double Truck

For negative moments and reactions at piers, a third condition is also considered. Two design trucks are applied, with a minimum headway between the front and rear axles of the two trucks equal to 50 feet. The rear axle spacing of the two trucks is set at a constant 14 feet. 90% of the effect of the two design trucks is combined with 90% of the design lane load, as presented in Figure 17.2-4. This loading is described in **LRFD [3.6.1.3.1]**.

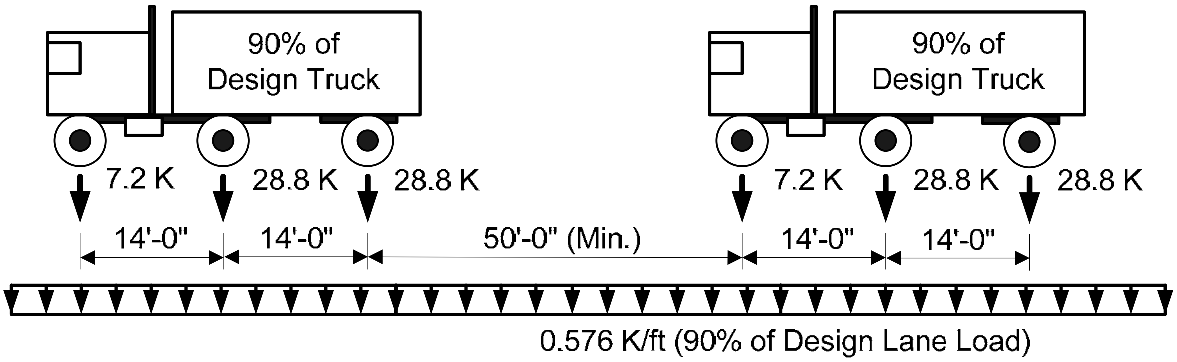


Figure 17.2-4
Double Truck

17.2.4.2.5 Fatigue Truck

The fatigue truck consists of one design truck similar to that described in 17.2.4.2.1 but with a constant spacing of 30 feet between the 32-kip axles, as presented in Figure 17.2-5. The fatigue truck is described in **LRFD [3.6.1.4.1]**.

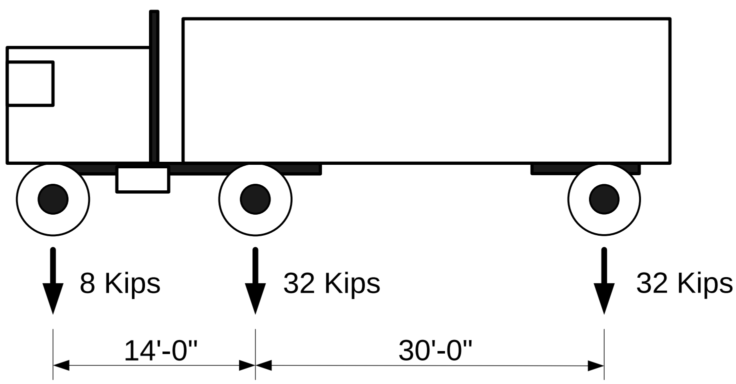


Figure 17.2-5
Fatigue Truck

17.2.4.2.6 Live Load Combinations

The live load combinations used for design are presented in Table 17.2-2.

Live Load Combination	Description	Reference
LL#1	Design tandem (+ IM) + design lane load	**LRFD [3.6.1.3.1]**
LL#2	Design truck (+ IM) + design lane load	**LRFD [3.6.1.3.1]**

17.2.9 Distribution of Dead Load to Substructure Units

For abutment design, the composite dead loads may be distributed equally between all of the girders, or uniformly across the slab.

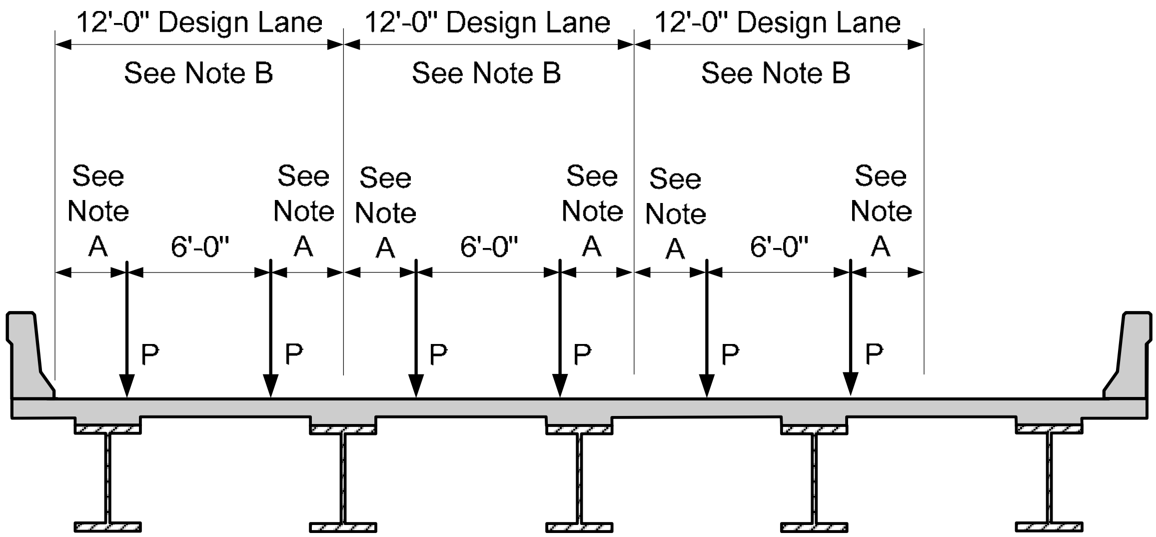
For pier design, the composite dead loads should be distributed equally between all of the girders, or uniformly across the slab, except for bridges with raised sidewalks. For girder bridges with raised sidewalks, follow the aforementioned Design Case 1 & 3 used for exterior girder design. For slab bridges with raised sidewalks, use the loading specified in Live Load Case 1 for exterior strips.

It is acceptable to consider the concrete diaphragm loads to be divided equally between all of the girders and added as point loads to the girder reactions.

17.2.10 Distribution of Live Loads to Substructure Units

See 17.2.9 for additional live load guidance regarding bridges with raised sidewalks. In the transverse direction, the design truck and design tandem should be located in such a way that the effect being considered is maximized. However, the center of any wheel load must not be closer than 2 feet from the edge of the design lane. The transverse live load configuration for a design truck or design tandem is illustrated in Figure 17.2-20. Pedestrian live load may be omitted if trying to maximize positive moment in a multi-columned pier cap.

As a reminder, always be aware to apply loads correctly. For example, for continuous spans the loading to the pier originates from the live load reaction rather than the sum of the live load shears of adjacent spans.



P = Wheel Load

Note A: Position wheel loads within the design lane such that the effect being considered is maximized; minimum = 2'-0".

Note B: Position design lanes across the roadway such that the effect being considered is maximized.

Figure 17.2-20

Transverse Configuration for a Design Truck or Design Tandem

Similarly, the design lane is distributed uniformly over the 10-foot loaded width. Since the design lane is 0.64 kips per linear foot in the longitudinal direction and it acts over a 10-foot width, the design lane load is equivalent to 64 psf. Similar to a design truck or design tandem, the 10-foot loaded width is positioned within the 12-foot design lane such that the effect being considered is maximized, as illustrated in Figure 17.2-21. The 10-foot loaded width may be placed at the edge of the 12-foot design lane.

Design Case 3	Dead and vehicle live loads	Strength I	At design section for overhang

Table 17.6-1
Deck Overhang Design Cases

The design load for Design Case 1 is a horizontal vehicular collision force, as illustrated in Figure 17.6-2. The transverse vehicle impact force, F_t, is specified in **LRFD [Table A13.2-1]** for various railing test levels. The force values specified in **LRFD [Table A13.2-1]** represent the total force, and neither dynamic load allowance nor multiple presence factors should be appleid to these values.

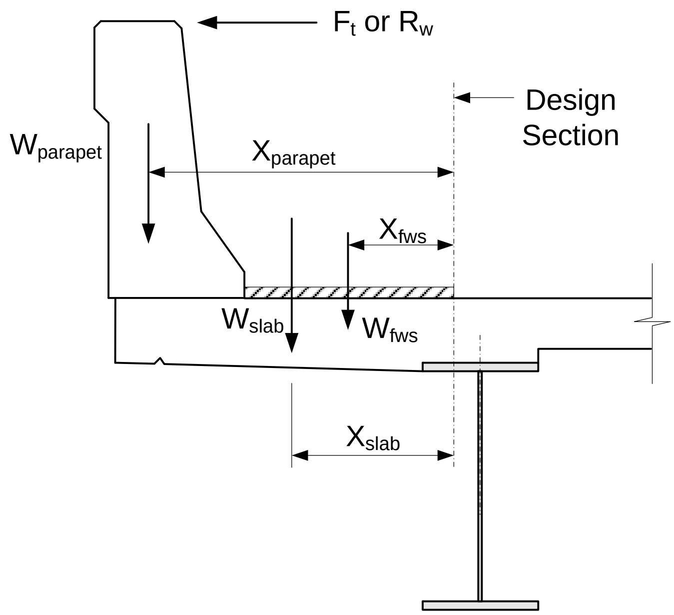


Figure 17.6-2
Design Case 1

The concrete barrier resistance, R_w, and the critical length of wall failure, L_c, are calculated in accordance with **LRFD [A13.3.1]**.

The longitudinal distribution length of the collision force for a continuous concrete barrier is calculated as illustrated in Figure 17.6-3. An angle of 30^o is conservatively assumed for the load distribution form the front face of the barrier to the overhang design section.

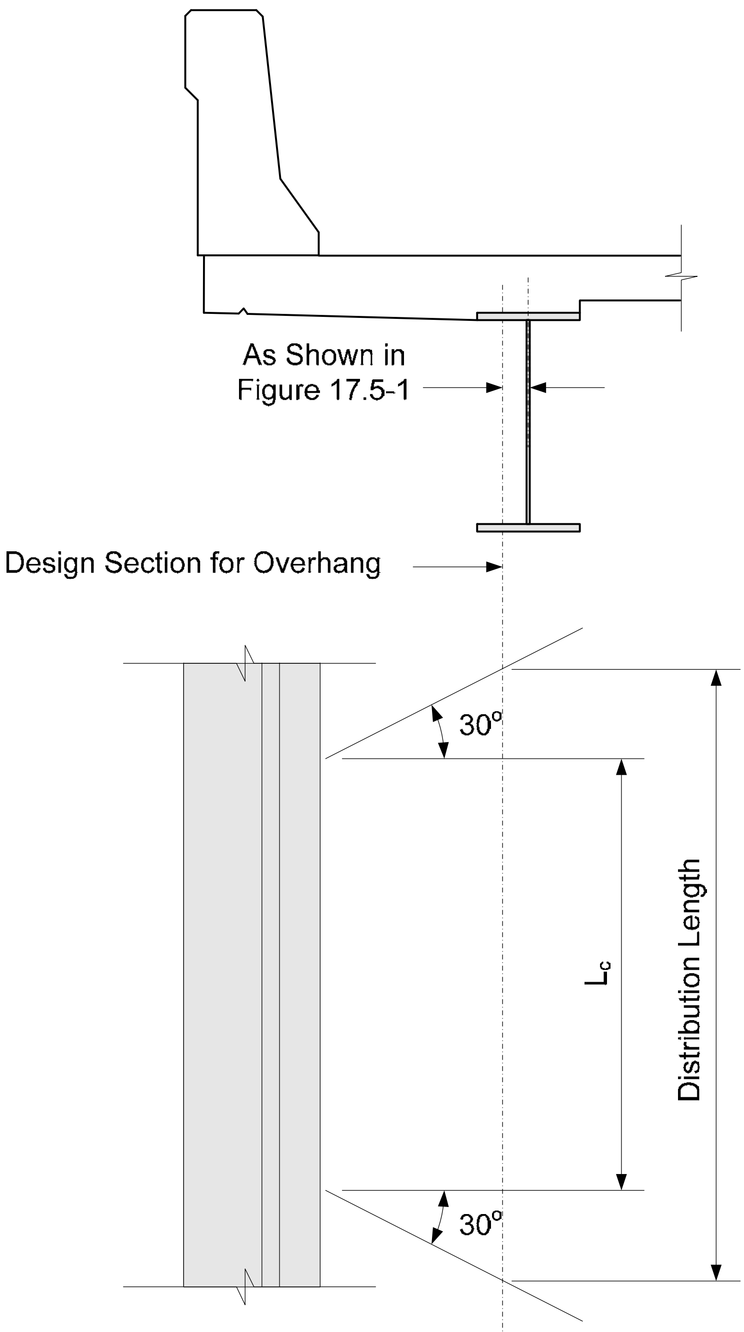


Figure 17.6-3
Assumed Distribution of Collision Moment Load in the Overhang

The design load for Design Case 2 is a vertical vehicular collision force, as illustrated in Figure 17.6-4. The vertical design force, F_v, is specified in **LRFD [Table A13.2-1]** for various railing test levels. The values for F_v specified in **LRFD [Table A13.2-1]** represent the total force, and neither dynamic load allowance nor multiple presence factors should be applied to these values.

17.10 Design of Precast Prestressed Concrete Deck Panels

17.10.1 General

An advantage of stay-in-place forms is that they can be placed in less time than it takes to place the forms for a conventional deck. There is also a labor savings because the extra step of removing deck forms is not required. Stay-in-place forms are often the preferred system for shallow box girders because of the difficulty of removing forms in a confined space.

If not detailed in the contract documents, precast concrete deck panels may be used at the option of the contractor, provided the specifications permit their use. A standardized special provision (STSP) for optional use of precast prestressed concrete deck plans is available from the Bureau of Highway Construction, Standards Development Section.

When a contractor elects to use precast deck panels at their option, the contractor is responsible for the shop plans of the panels and any other changes that may be required to the reinforcing steel in the cast-in-place portion of the deck. Payment to a contractor who chooses to use stay-in-place forms is based on the contract prices bid for the conventional cast-in-place deck.

Deck panels are only used between the inside faces of the exterior girders. The overhangs outside the exterior girders are formed and the concrete placed in the same way as in a conventional cast-in-place deck. On skewed decks, the contractor may form and cast the skewed portion of the deck full depth or they may use skewed end deck panels which may be individually precast or saw-cut from square end planks.

A problem with decks formed with concrete deck panels is that cracks often form in the cast-in-place concrete over the transverse joints between panels and along the edges of the panels parallel to the girders. Reflection cracking is less of a problem when these panels are used on prestressed concrete girders than on steel girders. Simple-span prestressed concrete girder bridges have less reflective cracking than continuous-span prestressed concrete girder bridges.

17.10.2 Deck Panel Design

The design of precast prestressed concrete deck panels shown in Table 17.10-1 is based on *AASHTO LRFD* design criteria. These panels were designed for flexure due to the HL-93 design truck live load, dead load of the plastic concrete supported by the panels, a construction load of 50 psf, dead load of the panels and a future wearing surface of 20 psf. The live load moments were obtained from **LRFD [Table A4-1]**.

At the request of precast deck panel fabricators, only two strand sizes are used – 3/8 inch and 1/2 inch. Precast deck panel fabricators do not want the additional overhead expense of stocking 7/16-inch strand. Strand spacing is given in multiples of 2 inches.

WisDOT exception to AASHTO:

A 3-inch minimum panel thickness is used, even though **LRFD [9.7.4.3.1]** specifies a minimum thickness of 3.5 inches.

The decision to use a 3-inch minimum panel was based on the successful use of 3-inch panels by other agencies over many years. In addition, a minimum of 5 inches of cast-in-place concrete is preferred for crack control and reinforcing steel placement. A 3.5-inch panel thickness would require an 8.5-inch deck, which would not allow direct substitution of panels for a traditionally designed 8-inch deck.

A study performed at Iowa State University determined that a 3-inch thick panel with coated 3/8-inch strands at midthickness spaced at 6 inches, along with epoxy-coated 6 x 6 – D6 x D6 welded wire fabric, was adequate to prevent concrete splitting during strand detensioning. The use of #3 bars placed perpendicular to the strands at 9” spacing also prevents concrete splitting.

Panel thicknesses were increased by ½ inch whenever a strand spacing of less than 6 inches was required. Strands with a ½-inch diameter were used in panels 3½ inches thick or greater when 3/8-inch strands spaced at 6 inches were not sufficient.

The allowable tensile stress in the panels, as presented in **LRFD [Table 5.9.4.2.2-1]**, is as follows:

$$0.0948\sqrt{f'_c}$$

This allowable tensile stress limit is based on f'_c in units of ksi and is for components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions.

The transfer length of the strands is assumed to be 60 strand diameters at a stress of 202.5 ksi. The development length, L_d, of the strands, as presented in **LRFD [5.11.4.2]**, is assumed to be as follows:

$$L_d = k\left(f_{ps} - \frac{2}{3}f_{pe}\right)d_b$$

Where:

k = 1.0 for pretensioned members with a depth less than 24 inches

d_b = Nominal strand diameter (inches)

f_{ps} = Average stress in prestressing steel at the time when the nominal resistance of the member is required (ksi)

f_{pe} = Effective stress in prestressing steel after losses (ksi)

L_d = Development length beyond critical section (inches)

The minimum panel width is the length required for the panel to extend 4" onto the top flange as shown in Table 17.10-1. A linear reduction in f_{pe} is required if the panel width is less than two times the development length. The values shown in Table 17.10-1 consider this linear reduction.

The designs in Table 17.10-1 are based on uncoated prestressing strands. Grit-impregnated, epoxy-coated strands cost four times as much as uncoated strands but require about half the transfer and development length as uncoated strands. A cover of 1 1/4 inches is adequate to provide protection from chlorides for uncoated strands using a 5 ksi concrete mix. However, for bridges with high traffic volume, a 6 ksi mix is recommended.

LRFD [9.7.4.3.2] specifies that the strands need not extend beyond the panels into the cast-in-place concrete above the beams. This simplifies construction of the panels at the plant since they can be saw cut to the required length. Installation in the field is also simplified because extended strands may interfere with girder shear connectors. As a substitute for the strands that don't extend out of the panels, #4 bars spaced at twice the spacing of the transverse bars are placed on top of the panels over the girders in the cast-in-place concrete. These bars anchor the panels together to prevent or reduce longitudinal cracking over the ends of the panels and also resist any positive continuity moments that may develop. Also by not extending the strands into the cast-in-place concrete, the uncoated strands are not exposed to chlorides that may seep through cracks that may develop in the cast-in-place concrete.

LRFD [5.7.3.3.2] requires that the moment capacity of a flexural member be greater than 1.2 times the cracking moment based on the modulus of rupture. This requirement may be waived if the moment capacity is greater than 1.33 times the factored design moment. The purpose of this requirement is to provide a minimum amount of reinforcement in a flexural member so that a flexural failure will not be sudden or occur without warning. Tests have shown that for slabs on girders, the failure mode is a punching shear failure and not a flexural failure. ACI 10.5.4 also recognizes the difference between slabs and beams and does not require the same minimum reinforcement for slabs. For these reasons, **LRFD [5.7.3.3.2]** was not considered in the designs of the panels shown in Table 17.10-1. However, panels with a width of 6 feet or more meet the requirements of **LRFD [5.7.3.3.2]**.

17.10.3 Transverse Reinforcement for Cast-in-Place Concrete on Deck Panels

The design of the transverse reinforcing steel in the cast-in-place concrete placed on deck panels is based on *AASHTO LRFD*. The live load moments used to determine the size and spacing of the transverse reinforcing bars placed in the top of the cast-in-place concrete are from **LRFD [Table A4-1]**. The reinforcing steel in the cast-in-place concrete is also designed for a future wearing surface of 20 psf. With stay-in-place forms, there are no negative moments from the dead load of the cast-in-place concrete. The required reinforcing steel shown in Table 17.10-2 is based on both the strength requirement and crack control requirement.

Crack control was checked in accordance with **LRFD [5.7.3.4]** and as shown in 17.5.3.1. A concrete strength of 4 ksi was assumed, and the haunch height over the girders was not considered.

The distance from the centerline of the girder to the design section is from **LRFD [4.6.2.1.6]**. For prestressed concrete girders, use the values in Figure 17.5-1.

The reinforcing steel in Table 17.10-2 does not account for deck overhangs. However, Table 17.6-2, Table 17.6-3, Table 17.6-4 and Table 17.6-5 provide the minimum reinforcing steel required in the overhangs. Also for any portion of a deck not supported by deck panels, use Table 17.5-1 for determining the required reinforcing steel.

17.10.3.1 Longitudinal Reinforcement

For continuous prestressed concrete girders, the longitudinal reinforcing steel over the piers is the same as that required for a conventional deck. For steel girders, see 17.5.3.2 for longitudinal continuity reinforcement.

17.10.4 Details

Precast deck panels should extend a minimum of 1.5 inches beyond the face of concrete diaphragms at the substructure units. The transverse joints between panels in adjacent bays should be staggered, preferably a distance about 1/2 panel length. Staggering the joints helps to minimize transverse reflective cracking.

Panels should never rest directly on a girder flange. According to **LRFD [9.7.4.3.4]**, "The ends of the formwork panels shall be supported on a continuous mortar bed or shall be supported during construction in such a manner that the cast-in-place concrete flows into the space between the panel and the supporting component to form a concrete bedding." The minimum width of bearing on the flange of a girder for both concrete and mortar or grout support is 3 inches. See Figure 17.10-1 and Figure 17.10-2 for additional information.

High-density expanded polystyrene is used to support the panels prior to the placement of the cast-in-place concrete under the panel. The polystyrene is cut to the required haunch height so a constant slab thickness is maintained. High-density expanded polystyrene is available in different strengths, and it is the responsibility of the contractor to determine the strength required based on the vertical load that must be resisted. Fiber board or sheathing panel supports are not allowed because the slight deflection of polystyrene compresses the concrete underneath the panel and results in less reflective longitudinal cracking along the panel edge.

When panels are supported on grout, the main function of the polystyrene is to form the haunch height and to form a dam for the grout placement. The grout must be placed immediately before placement of the panels. It is important that enough grout be placed so that the vertical load from the panels is supported by the grout and not by the polystyrene.

Some agencies specify a maximum haunch height. When it is exceeded, they allow the contractor to thicken the slab. Wisconsin does not specify a maximum haunch height and leaves that decision to the designer, who is better informed to make that decision based on the specific situation of their project.

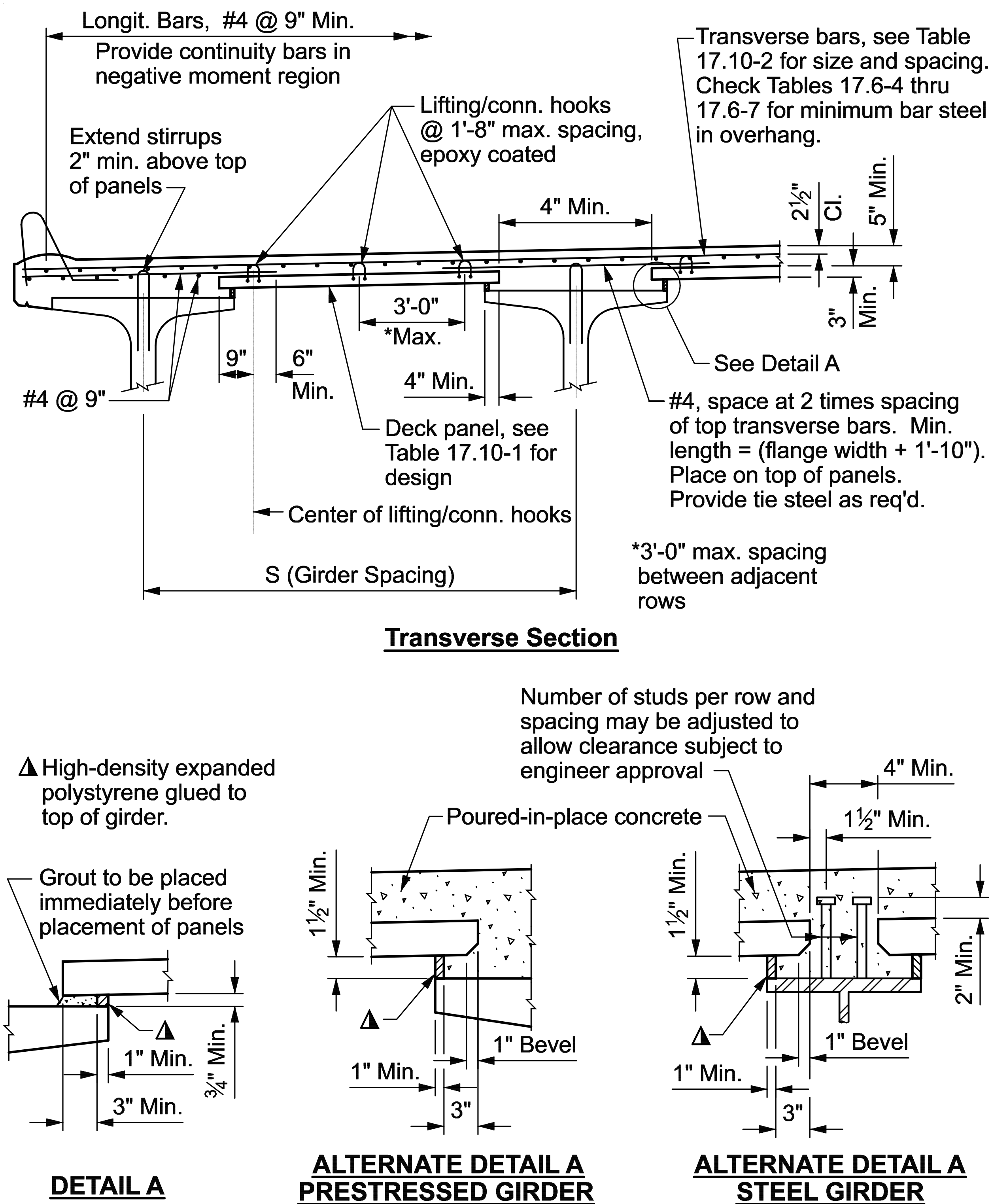
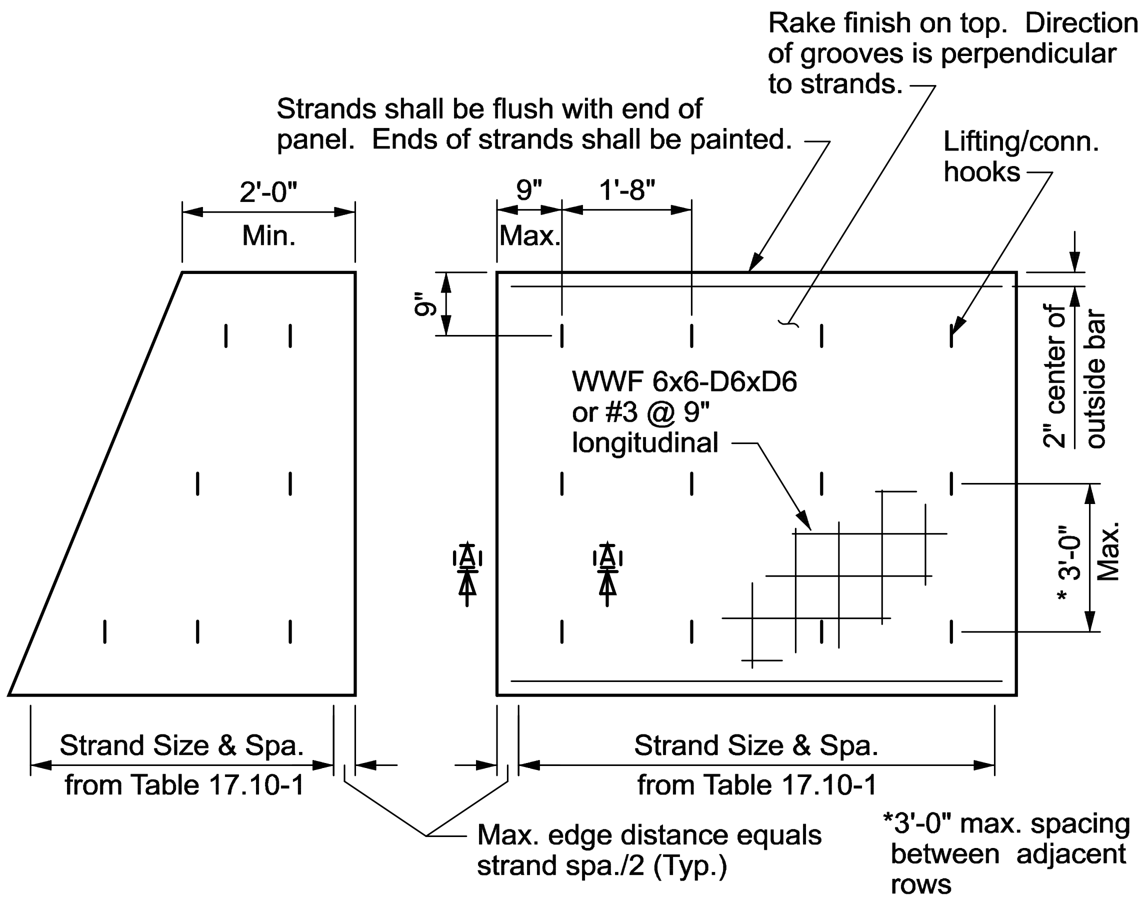
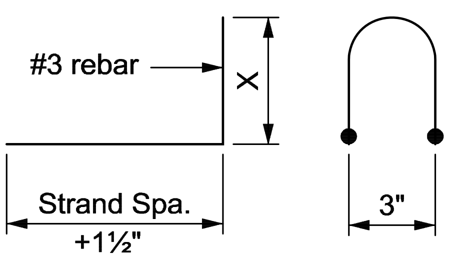


Figure 17.10-1
Transverse Section through Slab on Girders with Deck Panel and Details



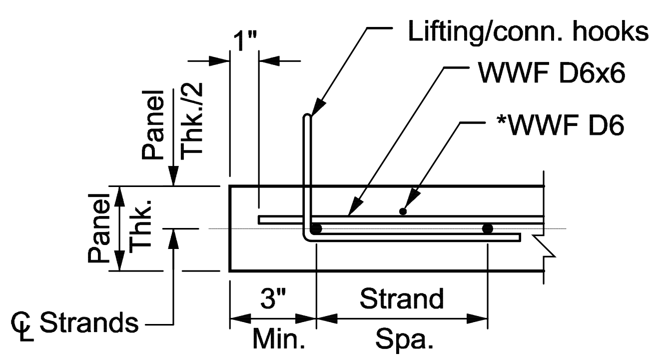
END SKEW DECK PANEL

DECK PANEL



Panel Thk. (in.)	X (in.)
3, 3½	4½
4	5
4½ , 5, 5½	5½

Lifting/Conn. Hook Detail



Part Section A-A

*Bars in WWF which are parallel to the strands must be a minimum of 1" clear from the strands.

Figure 17.10-2
Deck Panel Details

Girder Spacing "S"	Panel Thick. (Inches)	Total Slab Thick. (Inches)	Top Flange Width (Inches)												
			12		16		18		24		30		30		
			Strand		Strand		Strand		Strand		Strand		Strand		
			Spa. Inches	Pi Kips	Spa. Inches	Pi Kips	Spa. Inches	Pi Kips	Spa. Inches	Pi Kips	Spa. Inches	Pi Kips	Spa. Inches	Pi Kips	
4'-6"	3	8	10	13.17	10	12.33	10	11.92	10	11.08	10	11.08	10	11.08	3/8" Diameter Strands
4'-9"	3	8	10	13.58	10	12.75	10	12.75	10	11.50	10	11.08	10	11.08	
5'-0"	3	8	10	14.42	10	13.58	10	13.17	10	12.33	10	11.08	10	11.08	
5'-3"	3	8	10	14.83	10	14.00	10	13.58	10	12.75	10	11.92	10	11.08	
5'-6"	3	8	10	15.67	10	14.83	10	14.42	10	13.17	10	12.33	10	11.08	
5'-9"	3	8	10	16.50	10	15.67	10	15.25	10	14.00	10	13.17	10	11.50	
6'-0"	3	8	8	14.25	10	16.50	10	16.08	10	14.83	10	13.58	10	11.50	
6'-3"	3	8	8	15.45	8	14.25	10	16.92	10	15.67	10	14.42	10	11.92	
6'-6"	3	8	8	16.12	8	15.45	8	14.78	10	16.50	10	15.25	10	12.33	
6'-9"	3	8	8	17.12	8	16.12	8	15.78	8	14.25	10	16.08	10	13.17	
7'-0"	3	8	6	14.19	8	17.12	8	16.45	8	15.45	8	14.25	10	13.58	
7'-3"	3	8	6	14.94	6	14.19	6	13.62	8	16.12	8	15.12	10	14.42	
7'-6"	3	8	6	15.69	6	14.94	6	14.69	8	17.12	8	15.78	10	15.25	
7'-9"	3	8	6	16.44	6	15.69	6	15.44	6	14.44	8	16.78	10	16.50	
8'-0"	3	8	6	17.19	6	16.44	6	16.19	6	15.19	6	14.19	8	14.25	
8'-3"	3.5	8.5	6	16.76	6	16.01	6	15.76	6	14.76	6	13.47	8	14.14	
8'-6"	3.5	8.5	10	29.48	6	16.76	6	16.51	6	15.51	6	14.51	8	14.97	
8'-9"	3.5	8.5	8	26.44	10	30.06	10	29.06	6	16.26	6	15.26	8	15.97	
9'-0"	3.5	8.5	8	27.44	8	26.44	8	26.10	6	17.01	6	16.01	8	16.64	
9'-3"	3.5	8.5	8	28.77	8	27.77	8	27.10	10	30.06	6	16.76	6	14.01	
9'-6"	4	9	8	27.76	8	26.76	8	25.95	10	29.22	6	16.37	8	17.20	
9'-9"	4	9	8	29.09	8	27.76	8	27.43	10	30.62	6	17.12	6	14.37	
10'-0"	4	9	8	30.09	8	29.09	8	28.43	8	27.09	10	30.20	6	15.12	
10'-3"	4	9	6	25.48	8	30.09	8	29.76	8	28.09	8	26.76	6	15.87	
10'-6"	4	9	6	26.23	6	25.48	8	30.76	8	29.09	8	27.76	6	16.62	
10'-9"	4	9.5	6	26.73	6	25.73	6	25.23	8	29.43	8	27.76	6	16.12	
11'-0"	4	9.5	6	27.48	6	26.73	6	26.23	8	30.43	8	28.76	6	16.87	
11'-3"	4	9.5	6	28.48	6	27.48	6	26.98	6	25.73	8	30.09	10	30.20	1/2" Diameter Strands
11'-6"	4	9.5	6	29.48	6	28.48	6	27.98	6	26.73	6	25.23	8	25.95	
11'-9"	4	10	6	30.23	6	28.98	6	28.48	6	26.98	6	25.48	8	25.95	
12'-0"	4.5	10	6	29.62	6	28.62	6	28.12	6	26.62	6	25.37	8	26.50	
12'-3"	4.5	10	6	30.62	6	29.62	6	29.12	6	27.62	6	26.12	8	27.83	
12'-6"	5	10	6	30.34	6	29.34	6	28.84	6	27.59	6	26.34	8	28.28	
12'-9"	5	10.5	6	30.59	6	29.59	6	29.09	6	27.59	6	26.34	8	27.95	
13'-0"	5.5	10.5	6	30.36	6	29.36	6	29.11	6	27.61	6	26.36	8	28.77	
13'-3"	5.5	10.5	4	23.52	6	30.36	6	29.86	6	28.61	6	27.36	8	29.77	
13'-6"	5.5	10.5	4	24.18	4	23.52	4	23.18	6	29.36	6	28.11	8	30.77	

13'-9"	6	11	4	23.39	6	30.41	6	30.16	6	28.66	6	27.41	8	29.96	
14'-0"	6	11	4	24.06	4	23.39	4	23.06	6	29.66	6	28.16	8	30.96	

Notes:

- Designed per AASHTO LRFD Specifications with HL 93 Loading.
- f’c = 6.0 ksi
- f’ci = 4.4 ksi
- f’c slab = 4.0 ksi
- f’s = 270 ksi (low relaxation)
- Design loading includes 20 psf for future wearing surface and 50 psf for construction load. Pi’s in Table are a minimum and may be increased to a maximum of 0.75 x f_s x A_s. Strands are located at the centroid of the panels.

Table 17.10-1
Precast Prestressed Concrete Deck Panel Design Table

Girder Spacing "S"	Total Slab Thick. Inches	Distance From C/L of Girder to Design Section (Inches)					
		3	4	5	6	10	15
4'-6"	8	#4 @ 9	#4 @ 9.5	#4 @ 10	#4 @ 10	#4 @ 11.5	#4 @ 12.5
4'-9"	8	#4 @ 8	#4 @ 8.5	#4 @ 9	#4 @ 9.5	#4 @ 11	#4 @ 12.5
5'-0"	8	#4 @ 7	#4 @ 7.5	#4 @ 8	#4 @ 8.5	#4 @ 10.5	#4 @ 12.5
5'-3"	8	#4 @ 6.5	#4 @ 7	#4 @ 7.5	#4 @ 8	#4 @ 10	#4 @ 12
5'-6"	8	#4 @ 6.5	#4 @ 6.5	#4 @ 7	#4 @ 7.5	#4 @ 9.5	#4 @ 12
5'-9"	8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 7	#4 @ 9	#4 @ 11.5
6'-0"	8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 8.5	#4 @ 11
6'-3"	8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 8.5	#4 @ 11
6'-6"	8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 8	#4 @ 10.5
6'-9"	8	#4 @ 6	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 7.5	#4 @ 10.5
7'-0"	8	#5 @ 7.5	#5 @ 8	#4 @ 6.5	#4 @ 6.5	#4 @ 7	#4 @ 10
7'-3"	8	#5 @ 7.5	#5 @ 7.5	#4 @ 6	#4 @ 6.5	#4 @ 6.5	#4 @ 10
7'-6"	8	#5 @ 7.5	#5 @ 7.5	#4 @ 6	#4 @ 6.5	#4 @ 6.5	#4 @ 9.5
7'-9"	8	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#4 @ 6.5	#4 @ 6.5	#4 @ 8.5
8'-0"	8	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#4 @ 6	#4 @ 6.5	#4 @ 8

8'-3"	8.5	#5 @ 7.5	#5 @ 8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 8
8'-6"	8.5	#5 @ 7.5	#5 @ 8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 8
8'-9"	8.5	#5 @ 7.5	#5 @ 7.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 7.5
9'-0"	8.5	#5 @ 7.5	#5 @ 7.5	#5 @ 8	#4 @ 6.5	#4 @ 6.5	#4 @ 7.5
9'-3"	8.5	#5 @ 7.5	#5 @ 7.5	#5 @ 8	#4 @ 6.5	#4 @ 6.5	#4 @ 7
9'-6"	9	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8.5	#5 @ 9.5	#4 @ 7
9'-9"	9	#5 @ 7.5	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 9.5	#4 @ 7
10'-0"	9	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 8	#5 @ 9	#4 @ 6.5
10'-3"	9	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 8.5	#5 @ 9.5
10'-6"	9	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8.5	#5 @ 9
10'-9"	9.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 8.5	#5 @ 9
11'-0"	9.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 8.5	#5 @ 9
11'-3"	9.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 9
11'-6"	9.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 8	#5 @ 9
11'-9"	10	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 8	#5 @ 9
12'-0"	10	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 9
12'-3"	10	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 8	#5 @ 8.5
12'-6"	10	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8.5
12'-9"	10.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8.5
13'-0"	10.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 8	#5 @ 8.5
13'-3"	10.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8.5
13'-6"	10.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8.5
13'-9"	11	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 8	#5 @ 8
14'-0"	11	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8

Notes:

- Designed per AASHTO LRFD with HL-93 Loading.
- f'c deck = 4.0 ksi
- fy = 60 ksi
- Steel is 2 ½" clear from top of slab. Designed for 20 psf future wearing surface. "Total Slab Thickness" includes thickness of deck panel and poured in place concrete.

Table 17.10-2

Transverse Reinforcing Steel for Deck Slabs on Precast Concrete Deck Panels

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

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

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

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

Where:

K_1 = Fraction of concrete strength available to resist interface shear as specified in **LRFD [5.8.4.3].** 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 [5.8.4.3].** 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 [5.8.4.4]**

19.3.3.16 Web Shear Reinforcement

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

WisDOT policy item:

Web shear reinforcement shall be designed by **LRFD [5.8.3.4.3]** (Simplified 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:

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

Where:

A_v = Area of transverse reinforcement within distance, s (in^2)

V_n = Nominal shear resistance (kips)

V_c = Nominal shear resistance provided by tensile stress in 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 [5.8.2.9]** (in)

b_v = Minimum web width within depth, d_v

$\cot\theta$ shall be taken as follows:

- When $V_{ci} < V_{cw}$, $\cot\theta = 1.0$
- When $V_{ci} > V_{cw}$, $\cot\theta = 1.0 + 3\left(\frac{f_{pc}}{\sqrt{f'_c}}\right) \le 1.8$

$$V_u = 1.25DC + 1.5DW + 1.75(LL + IM)$$

$$V_n = V_u / \phi$$

Where:

V_u Strength I Limit State shear force (kips)

ϕ = 0.90 per **LRFD [5.5.4.2.1]**

See 17.2 for further information regarding load combinations.

Per **LRFD [5.8.3.4.3]**, determine V_c as the minimum of either V_{ci} or V_{cw} given by:

$$V_{cw} = (0.06\sqrt{f'_c} + 0.30f_{pc})b_v d_v + V_p$$

$$V_{ci} = 0.02\sqrt{f'_c}\,b_v d_v + V_d + \frac{V_i M_{cre}}{M_{max}} \ge 0.06\sqrt{f'_c}\,b_v d$$

Where:

f_{pc} = Compressive stress in concrete, after all prestress losses, at centroid of cross section resisting externally applied loads or at the web-flange junction when the centroid lies within the flange. (ksi) In a composite member, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the web-flange junction, due to both prestress and moments resisted by the member acting alone.

V_d = Shear force at section due to unfactored dead loads (kips)

V_i = Factored shear force at section due to externally applied loads occurring simultaneously with M_{max} (kips)

M_{cre} = Moment causing flexural cracking at the section due to externally applied loads (k-in)

M_{max} = Maximum factored moment at section due to externally applied loads (k-in)

$$V_i = V_u - V_d$$

$$M_{cre} = S_c\left(f_r + f_{cpe} - \frac{12M_{dnc}}{S_{nc}}\right)$$

$$M_{max} = M_u - M_{dnc}$$

Where:

S_c = Section modulus for the extreme tensile fiber of the composite section where the stress is caused by externally applied loads (in^3)

S_{nc} = Section modulus for the extreme tensile fiber of the noncomposite section where the stress is caused by externally applied loads (in^3)

f_{cpe} = Compressive stress in concrete due to effective prestress forces only, after all prestress losses, at the extreme tensile fiber of the section where the stress is caused by externally applied loads (ksi)

M_{dnc} = Total unfactored dead load moment acting on the noncomposite section (k-ft)

f_r = Modulus of rupture of concrete. Shall be $= 0.24\sqrt{f'_c}$ (ksi)

For a composite section, V_{ci} corresponds to shear at locations of accompanying flexural stress. V_{cw} corresponds to shear at simple supports and points of contraflexure. The critical computation for V_{cw} is at the centroid for composite girders.

Set the vertical component of the draped strands, V_p, equal to 0.0 when calculating V_n, as per **LRFD [5.8.3.3]**. This vertical component helps to reduce the shear on the concrete section. The actual value of V_p should be used when calculating V_{cw}. However, the designer may make the conservative assumption to neglect V_p for all shear resistance calculations.

WisDOT policy item:

Based on past performance, the upper limit for web reinforcement spacing, s_{max}, per **LRFD [5.8.2.7]** will be reduced to 18 inches.

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

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

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

Where:

$$\upsilon_u = \frac{V_u - \phi V_p}{\phi b_v d_v}$$ per **LRFD [5.8.2.9]**.

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 D16.

Per **LRFD [5.8.3.5]**, at the inside edge of the bearing area to the section of critical shear, the longitudinal reinforcement on the flexural tension side of the member shall satisfy:

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

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

19.3.3.17 Continuity Reinforcement

The design of non-prestressed reinforcement for negative moment at the support is based on the Strength I limit state requirements of **LRFD [5.7.3]**:

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

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.

<u>WisDOT exception to AASHTO:</u>

Composite sections formed by WisDOT standard prestressed concrete girders shall be considered to be tension-controlled for the design of the continuity reinforcement. The ε_t check, as specified in **LRFD [5.7.2.1]**, is not required, and $\phi = 0.9$.

<u>WisDOT policy item:</u>

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

<u>WisDOT policy item:</u>

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 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 $M_r \geq M_u$.

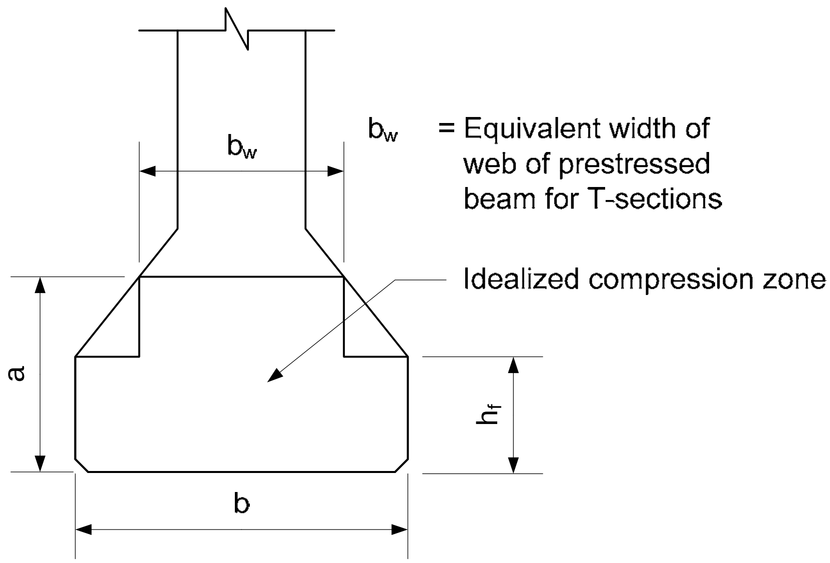

Figure 19.3-4
T-Section Compression Flange Behavior

The continuity reinforcement should also be checked to ensure that it meets the crack control provisions of **LRFD [5.7.3.4]**. 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 [5.14.1.4.8]**. 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 [C5.14.1.4.2]** 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 [5.14.1.4]** 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 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 girder with a draped strand profile.

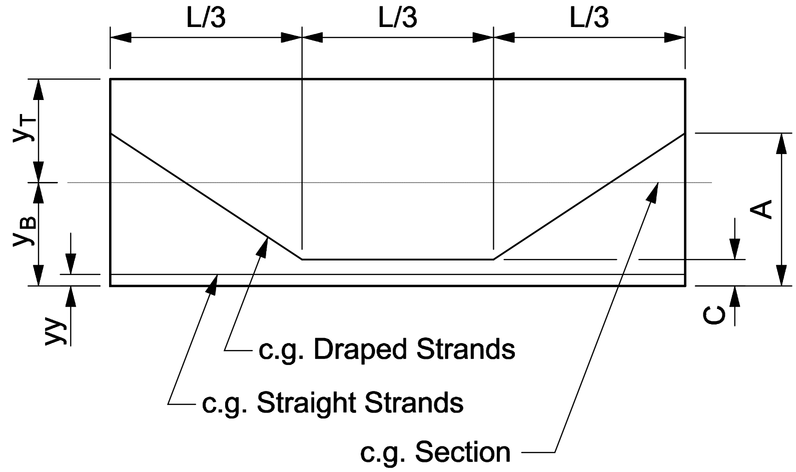

Figure 19.3-5
Typical Draped Strand Profile

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:

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

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

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

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

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:

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

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:

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

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

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 due to the dead load of the deck and 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 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.

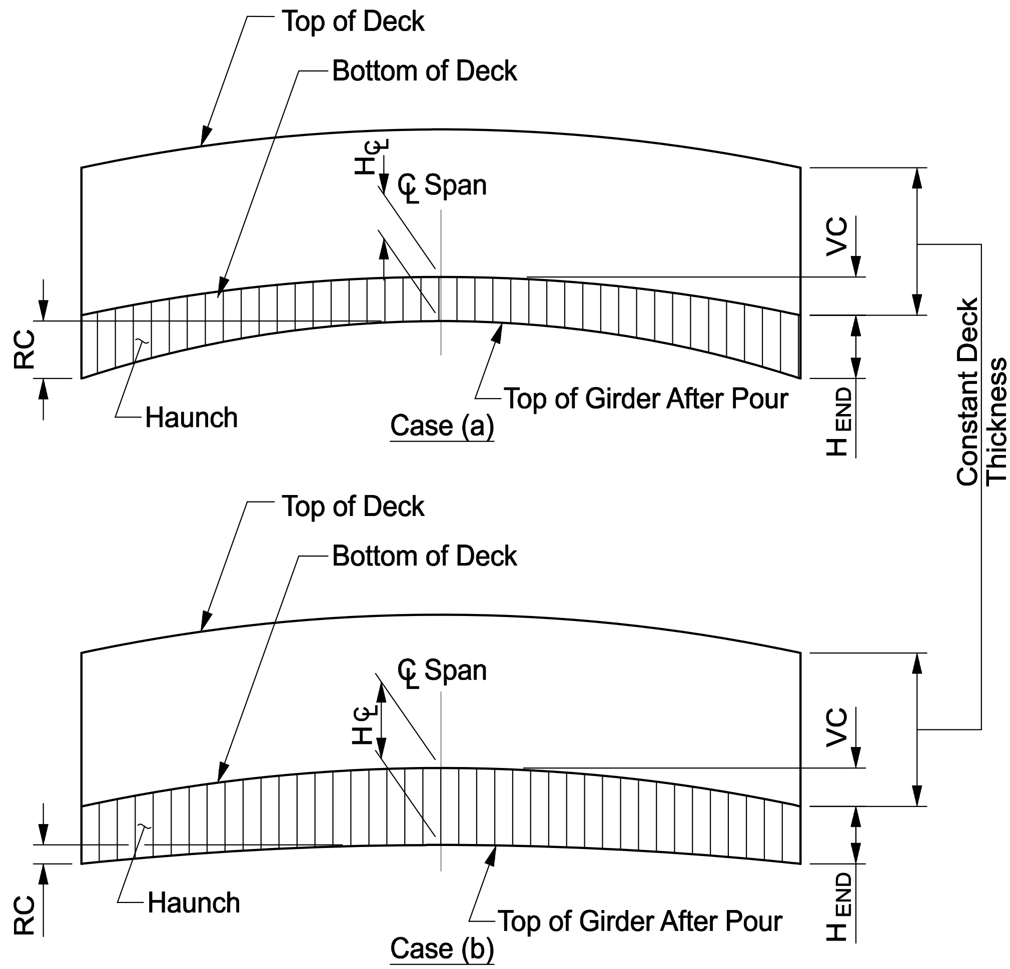

Figure 19.3-6
Relationship Between Top of Girder and Bottom of Deck

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 concrete I-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 Girder Sections

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

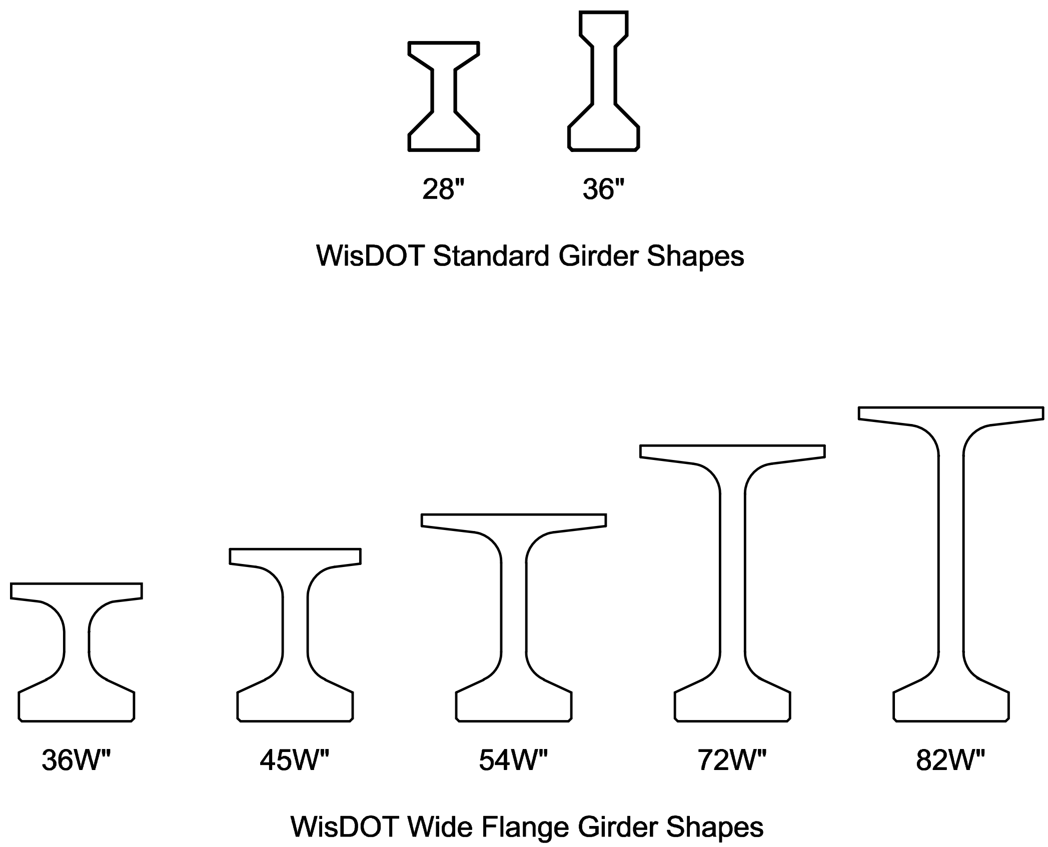

Figure 19.3-7
I Girder Family Details

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 $0.75f_{pu}$, a concrete haunch of 2", 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 2 ½” was used for these girders in the following tables. **The haunch values currently used in all the tables are somewhat unconservative -- do not push the span limits/girder spacing during preliminary design.** See Table 19.3-2 for guidance regarding use of excessively long prestressed girders.

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

f'_c girder = 8,000 psi

f'_c slab = 4,000 psi

Haunch height = 2” or 2 ½”

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

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24.4 Design Considerations

Steel girder structures are analyzed and designed using LRFD. *AASHTO LRFD* provides the details for designing simple and continuous steel girders for various span lengths using LRFD.

WisDOT Policy Item:

Do no utilize optional **LRFD (Appendix A6)** providing Flexural Resistance of Straight Composite I-Sections in Negative Flexure and Straight Noncomposite I-Sections with Compact or Noncompact Webs.

Design considerations common to all superstructure types, including distribution of loads, dead load, traffic live load, pedestrian load and wind load, are presented in Chapter 17 – Superstructures - General.

24.4.1 Design Loads

24.4.1.1 Dead Load

For steel girder structures, dead loads should be computed based on the following:

1. The weight of the concrete haunch is determined by estimating the haunch depth at 2- 1/2" and the width equal to a weighted average of the top flange width.
2. The weight of steel beams and girders is determined from the AISC Manual of Steel Construction. Haunched webs of plate girders are converted to an equivalent uniform partial dead load.
3. The weight of secondary steel members such as bracing, shear studs and stiffeners can be estimated at 30 plf for interior girders and 20 plf for exterior girders.
4. A dead load of 20 psf carried by the composite section is added to account for a future wearing surface.

AASHTO LRFD specifies that the effect of creep is to be considered in the design of composite girders which have dead loads acting on the composite sections. As specified in **LRFD [6.10.1.1.1a]** and **LRFD [6.10.1.1.1b]**, for the calculation of the stresses in a composite girder, the properties of the steel section alone should be used for permanent loads applied before the concrete deck has hardened or is made composite. The properties of the long-term 3n composite section should be used for permanent loads applied after the concrete deck has hardened or is made composite. The properties of the short-term n composite section should be used for transient loads applied after the concrete deck is made composite. **LRFD [6.10.1.1.1d]** requires that n be used to compute concrete deck stresses due to all permanent and transient loads.

Information regarding dead load deflections is given in 24.4.8

24.4.1.2 Traffic Live Load

For information about LRFD traffic live load, see 17.2.4.2.

24.4.1.3 Pedestrian Live Load

For information about LRFD pedestrian live load, see 17.2.4.4.

24.4.1.4 Temperature

Steel girder bridges are designed for a coefficient of linear expansion equal to .0000065/°F at a temperature range from -30 to 120°F. Refer to Chapter 28 – Expansion Devices for expansion joint requirements, and refer to Chapter 27 – Bearings for the effect of temperature forces on bearings.

24.4.1.5 Wind

For information about LRFD wind load, see Chapter 17 – Superstructures - General. In addition, see 24.6.16 for wind effects on girder flanges and 24.6.22 for design of bracing.

24.4.2 Minimum Depth-to-Span Ratio

Traditional minimum depths for constant depth superstructures are provided in **LRFD [Table 2.5.2.6.3-1]**. For steel simple-span superstructures, the minimum overall depth of the composite girder (concrete slab plus steel girder) is 0.040L and the minimum depth of the I-beam portion of the composite girder is 0.033L. For steel continuous-span superstructures, the minimum overall depth of the composite girder (concrete slab plus steel girder) is 0.032L and the minimum depth of the I-beam portion of the composite girder is 0.027L. For trusses, the minimum depth is 0.100L.

For a given span length, a preliminary, approximate steel girder web depth can be determined by referring to Table 24.4-1. This table is based on previous design methods and should therefore be used for preliminary purposes only. However, it remains a useful tool for approximating an estimated range of web depths for a given span length. Recommended web depths are given for parallel flanged steel girders. The girder spacings and web depths were determined from an economic study, deflection criteria and load-carrying capacity of girders for a previous design method.

From a known girder spacing, the effective span is computed as shown in Figure 17.5-1. From the effective span, the slab depth and required slab reinforcement are determined from tables in Chapter 17 – Superstructures - General, as well as the additional slab reinforcement required due to slab overhang.

connections at the strength limit state subject to shear on the effective area is to be taken as follows:

$$Rr = 0.6\phi_{e2}F_{exx}$$

Where:

ϕ_{e2} = Resistance factor for shear on the throat of the weld metal in fillet welds specified in **LRFD [6.5.4.2]** (= 0.80)

F_{exx} = Classification strength of the weld metal (ksi) (for example, for E70 weld metal, F_{exx} = 70 ksi)

If a certain size fillet weld must be used in adjacent areas of a particular joint, it is desirable to use the same size weld to allow the same electrodes and welding equipment to be used for that joint and to simplify the inspection.

24.4.8 Dead Load Deflections, Camber and Blocking

Show the total dead load deflections, including composite dead load (without future wearing surface) acting on the composite section, at tenth points of each span. Distribute the composite dead load evenly to all girders and provide one deflection value for a typical interior girder. Chapter 17 – Superstructure-General illustrates three load cases for exterior girder design with raised sidewalks, cases that provide a conservative envelope to ensure adequate girder capacity. However, the above composite dead load distribution should be used for deflection purposes. Total deflections and deflections for concrete only are computed to the nearest 0.1” and shown on a deflection diagram.

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

When straight girder sections between splice joints are erected, final girder elevations usually vary in height between the girder and roadway elevations due to dead load deflections and vertical curves. Since a constant slab thickness is detailed, a concrete haunch between the girder and slab is used to adjust these variations. If these variations exceed 3/4", the girder is cambered to reduce the variation of thickness in the haunch. This is done for all new girders, including widenings. Straight line chords between splice points are sometimes used to create satisfactory camber. If separate deflections are required for exterior girders, as described in Chapter 6 – Plan Preparation and Chapter 17 – Superstructure-General, provide only one camber value for all girders that is a best fit.

Welded girders are cambered by cutting the web plates to a desired curvature. During fabrication, all web plates are cut to size since rolled plates received from the mill are not straight. There is a problem in fabricating girders that have specified cambers less than 3/4", so they are not detailed.

Rolled sections are cambered by the application of heat in order that less camber than recommended by AISC specifications may be used. The concrete haunch is used to control the remaining thickness variations.

A blocking diagram is given for all continuous steel girder bridges on a vertical curve. Refer to Standard for Blocking & Slab Haunch Details for blocking and slab haunch details. Blocking heights to the nearest 1/16" are given at all bearings, field splices and shop splice points. The blocking dimensions are from a horizontal base line passing through the lower end of the girder at the centerline of bearing.

The plans should show in a table the top of steel elevations after erection at each field splice and at the centerline of all bearings.

It should be noted that the plans are detailed for horizontal distances. The fabricator must detail all plates to the erected position considering dead loads. Structure erection considerations are three-dimensional, considering slope lengths and member rotation for member end cuts.

24.4.9 Expansion Hinges

The expansion hinge as shown on Standard for Expansion Hinge Joint Details is used where pin and hanger details were previously used. The expansion hinge is more redundant and, if necessary, the bearings can easily be replaced.

vertical. Also, inspection manholes are often inserted in the bottom flanges of closed-box sections near supports. These manholes should be subtracted from the bottom-flange area when computing the elastic section properties for use in the region of the access hole. If longitudinal flange stiffeners are present on the closed-box section, they are often included when computing the elastic section properties.

When investigating web bend-buckling resistance for closed-box sections, **LRFD [6.11.3.2]** states that the maximum longitudinal flange stress due to the factored loads, calculated without consideration of longitudinal warping, must not exceed $\phi_f F_{crw}$ at sections where non-composite box flanges are subject to compression during construction. For more information about the web bend-buckling resistance of box girders, refer to 24.12.1. In *AASHTO LRFD*, a box flange is defined as a flange connected to two webs.

Torsion in structural members is generally resisted through a combination of St. Venant torsion and warping torsion. For closed cross-sections such as box girders, St. Venant torsion generally dominates. Box girders possess favorable torsional characteristics which make them an attractive choice for horizontally curved bridges. However, due to redundancy concerns, use of single-box and two-box girder bridges should be avoided unless absolutely necessary.

WisDOT policy item:

Rigorous analysis of single-box and two-box girder bridges to eliminate the need for in-depth fracture critical inspections is not allowed.

24.16 Design Example

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30.1 Crash Tested Bridge Railings

All bridge railings must have passed the crash tests as recommended in the NCHRP report 350 for Bridge Railings. In order to use railings other than Bridge Office Standard railing details, the railings must conform to crash tested rails which are available from the FHWA office. Any railings that are not crash tested must be reviewed by FHWA when they are used on bridge, culvert, retaining wall, etc.

Railings must meet the criteria for TL-3 or greater to be used on all roadways. Railings meeting TL-2 criteria may be used on roadways where the speed is 45 mph or less.

30.2 Railing Application

The designation for railing types are shown on the Standards. Standard railing details are generally employed as follows:

1. The "LF" parapet is preferred on state and interstate highway bridges except for some limited short span structures. The "HF" parapet is used where there is high truck traffic and curved horizontal alignment creating more potential for overtopping the parapet. These parapets meet crash test criteria for TL-4.

   An LF or solid parapet is preferred on all grade separation structures and railroad crossings to minimize snow removal falling on the traffic below.

2. Type "H" aluminum or steel railing is used on top of either the Vertical Face Parapet "A" or sloped face parapets when required for pedestrians and/or bicyclists. Both situations, with or without the “H” rail, meet the criteria for TL-4. For a design speed greater than 45 mph, the sloped face parapet is recommended. If the structure has a sidewalk on one side only, the sloped face parapet is used on the side opposite of the sidewalk.

3. Type "F" steel railing is not allowed on the National Highway System (NHS). Type “F” railing may be used on non-NHS roadways with a design speed of 45 mph or less. This railing facilitates drainage and snow removal but is usually more expensive than the Sloped Face Parapet if drains are not required at the ends of the bridge. In order to meet AASHTO Specifications three or more posts are attached to the Type "F" railing. May be used when TL-2 criteria is required.

4. Type “M” steel railing is used on state maintained bridges where the Region requests an open railing. It is similar to the Type “F” but has a higher crash test rating. Used in place of the Type “W” rail on girder type structures. Meets criteria for TL-4.

5. Type "W" railings may be used on all functional classes of Wisconsin highway structures. Generally, Type "W" railing is considered when the highway approach requires standard beam guard and if the structure is 80 feet or less in length. Meets criteria for TL-3. The Type “W” rail shall only be used on concrete slab structures. The use of this railing on girder type structures shall be discontinued.

6. Aesthetic railings may be used if crash tested according to Section 30.1. The Texas style parapet, Type “TX”, has been crash tested but it is very expensive. Form liners to simulate the openings would reduce the cost of this parapet. Meets criteria for TL-2.

   The Standards show some Combination Railings, Type “C1” through “C6” that are approved as aesthetic railings attached to concrete parapets. The aesthetic additions are at least 5” from the crash-tested rail face and do not present a snagging potential. Meets criteria for TL-2, but should only be used when the design speed is 45 mph or less, or the railing is protected by a barrier between the roadway and sidewalk.

7. The “51F” parapet may only be used on the median side, when it provides a continuation of the approach 51 inch high median barrier.

8. The Type “PF” tubular railing is not allowed on the National Highway System (NHS). Type “PF” railing may be used on non-NHS roadways with a design speed of 45 mph or less. This railing is similar to the Type “F” railing with two main differences. The height of this rail meets the requirements for pedestrian facilities. This is a solid rail type that can be used on a grade separation structure. May be used when TL-2 criteria is required.

9. If a box culvert has beam guard railing across the structure, then the rail members shall have provisions for a Thrie Beam connection at the ends of the structure as shown on SDD 14 B 20 Standards. Railing is not required on box culverts if there is a clear zone as defined in *Facilities Development Manual 11-15-1*. Non-Traversable hazards or fixed objects should not be constructed or allowed to remain within the clear zone. When this is not feasible, the use of a traffic barrier to shield the hazard or obstacle may be warranted. The barrier shall be provided only when it is cost effective as defined in *Facilities Development Manual Procedure 11-45-1*.

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

11. Timber Railing as shown in the Standards is not allowed on the National Highway System (NHS). Timber Railing may be used on non – NHS roadways with a design speed of 45 mph or less. Meets criteria for TL-2.

12. Chain Link Fence and Ornamental Protective Screening, as shown in the Standards, may be attached to the top of concrete parapets (or directly to the deck if on a sidewalk separated from the roadway by a crashworthy barrier). Ornamental Protective Screening should only be used when the design speed is 45 mph or less, or the screening is protected by a barrier between the roadway and sidewalk. Chain Link Fence can be used for any design speed.

See the *Facilities Development Manual 11-40-1* for additional railing application requirements.

30.3 Design Details

1. Provide for expansion movement in tubular railings where expansion devices or concrete parapet deflection joints exist on the structure plan details. The tubular railing splice should be located over the joint at its midspan and made continuous with a movable internal sleeve. On conventional structures where expansion joints are likely to occur at the abutments only, if tubular railing is employed, the posts may be placed at equal increments providing that no post is nearer than 2 feet from deflection joints in the parapet at the piers.

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

3. Refer to Standard Detail, Vertical Face Parapet “A” for detailing concrete parapet or median deflection joints. These joints are used because of previous experience with transverse deck cracking beneath the parapet joints.

4. Horizontal cracking occurred near the top of some concrete parapets which were slip formed. Similar cracking has not occurred on parapets cast in forms. Therefore, slip forming of bridge parapets should not be allowed.

5. Detail erection joints at the one-sixth panel point for Type "F" railing. This location will insure primarily a shear transfer at the railing splices. For beam guard type railing, locate the expansion splice at a post or on either side of the expansion joint.

6. On skewed bridges where the length from the rail post to the first guard rail posts exceeds 3 feet, employ the following detail: Extend the railing to the back face of the abutment. Bolt a plate to the back of the rails right before the rail bend. In case of vehicle impact, this detail will cause the rails to act as a unit in preventing vehicle wheel snagging.

7. Note the AASHTO Specification for a maximum opening of 6 inches on lower rail elements.

8. Sidewalks - If there is a parapet between the roadway and a sidewalk and the roadway side of the parapet is more than 11’-0” from the exterior edge of deck, the sidewalk width must be 10’-0” clear between barriers. Access must be provided to the sidewalk for the “snooper truck” to inspect the underside of the bridge. The boom extension on most trucks does not exceed 11’-0” so provision must be made to get the truck closer to the edge.

9. The height of curbs for sidewalks is usually 6 inches. This height is more desirable than higher heights with regards to safety because it is less likely to vault vehicles.

30.4 Utilities

The maximum allowable conduits that can be placed in HF or LF parapets are shown in the following sketches (LF only shown). Junction (Pull) boxes can only be used with 2 inch diameter conduit. The maximum length of 3 inch conduit is 190 feet, as no boxes are allowed.

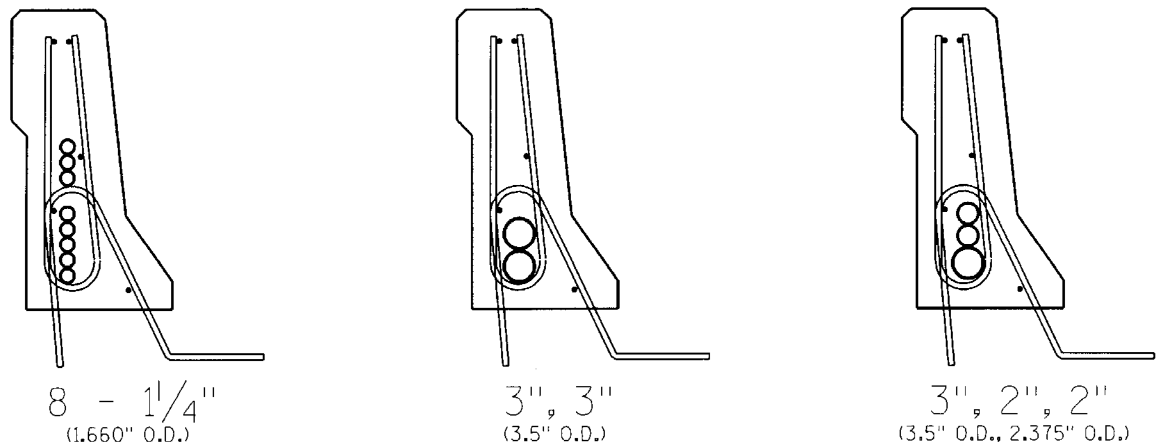

Figure 30.4-1
Maximum Allowable Conduits in HF and LF Parapets

When light poles are mounted on top of parapets and the design speed exceeds 45 mph the light pole must be located behind the back edge of the parapet. The poles should also be placed over the piers unless there is an expansion joint. Place 4 feet away if this is the case.

FDM 9-25-5 addresses whether a bench mark disk should be set on a structure. Structures are not usually preferred due to possible elevation changes from various causes. WisDOT has discontinued the statewide practice of furnishing a disk and requiring it to be placed on a structure. WisDOT Region Offices may continue to provide a bench mark for the contract to be set. Consult the Region Office to determine if a bench mark should be included in the plan set.

30.5 Protective Screening

Protective screening is a special type fence constructed on the sides of an overpass to discourage and/or prevent people from dropping or throwing objects onto vehicles passing underneath the structure. Protective screening is generally chain link type fencing attached to steel posts mounted on top of a vertical parapet or on a sidewalk surface. The top of the protective screening may be curved inward toward the structure, if mounted on a parapet and on a sidewalk, to prevent objects from being thrown off the overpass structure. Aesthetics is enhanced by using a colored protective screening which can be coordinated with the color of the structure. See Chapter 37 for screening details.

Examples of situations that warrant consideration of protective screening are:

1. If there is a history of or instances of objects being dropped or thrown from an existing overpass.

2. For all new overpasses if there have been instances at other existing overpasses in the area.

3. On overpasses near a school, playground, residential area or any other location where the overpass may be used by children who are not accompanied by an adult.

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

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

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

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

Occasionally access to light poles behind protective screening is required or the screening may need repair. To gain access, attach fence stretchers to the fencing and remove one vertical wire by threading or cutting. The vertical wire may be cut without using fence stretchers. To repair, attach fence stretchers and thread a vertical wire in place of the one removed by either reusing the one in place or using a new one.

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

30.6 Medians

The typical height of any required median curb is 6 inches. This will prevent normal crossovers and reduce vaulting on low speed roadways without excessive dead load. The preferred median for structures is shown on the Standard for Median and Raised Sidewalk Details.

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40.1 General

New bridges are designed for a minimally expected life of 75 years. Preliminary design considerations are site conditions, structure type, geometrics, and safety. Refer to Bridge Manual Chapter 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 five-year cycle now.

40.3 Bridge Replacements

Bridge rehabilitation is preferred over bridge replacement if the final structure provides adequate serviceability. In order to obtain federal funding eligibility for rehabilitation or replacement; the bridge must be Structurally Deficient or Functionally Obsolete. The Federal Sufficiency Number is a guide for federal participation which is required to be less than 50 for replacement. Also, Wisconsin DOT requires the Rate Score to be less than 75. Bridges are not eligible for replacement unless the Substructure or Superstructure Condition is 4 or less or the Inventory Rating is less than HS10 or the Alignment Appraisal is 4 or less.

A bridge becomes Structurally Deficient when the condition of the deck, superstructure or substructure is rated 4 or less; or when the inventory load capacity is less than 10 tons (89.0 kN); or when the waterway adequacy is rated a 2.

A bridge becomes Functionally Obsolete when the bridge roadway width, vertical clearance, or approach alignment is substandard (appraisal rating of 3 or less), or when the inventory load capacity is less than 15 tons; or when the waterway adequacy is rated a 3 or less.

Wisconsin DOT has established minimum roadway widths for bridges to remain in place on Rural, State and County Trunk Highways. As a minimum, bridge replacement is required for all bridges less than 100 ft. long and the useable width of the bridge is less than the following:

Design ADT	Rural Arterial Typically STH	Rural Arterial Typically CTH	Town Road
0-250	22’	20’	18’
251-750	22’	20’	22’
751-2000	Traveled Way + 2’	Traveled Way + 2’	Traveled Way + 4’
2001-4000	Traveled Way + 4’	Traveled Way + 4’	Traveled Way + 4’
Over 4001	Traveled Way + 6’	Traveled Way + 6’	Traveled Way + 4’

Table 40.3-1
Bridge Clear Width Rehabilitation Requirements

* If lane widening is planned as part of the 3R project, the usable bridge width should be compared to the planned width of the approaches after they are widened.

40.4 Rehabilitation Considerations

As a structure ages, rehabilitation is a necessary part of insuring some level of acceptable serviceability. The first consideration for any bridge rehabilitation decision is whether its geometry and load carrying capacity are sufficient to safely carry present and projected traffic. Information necessary to determine structure sufficiency includes structure inspection, inventory, traffic, maintenance, capacity and functional adequacy. The methods of reconstruction are based on the type of structures, existing condition or rating information, the preliminary details of rehabilitation, and traffic control costs. These are important factors in considering rehabilitation options such as either a deck protection system or a deck replacement.

The Regions are to evaluate bridge deficiencies when the bridge is placed in the program to insure that rehabilitation will remove all structural deficiencies. FHWA requires this review and Bureau of Structures (BOS) concurrence with all proposed bridge rehabilitation. On high cost bridges, a closer check of the Functionally Obsolete Criteria may be required. On high cost bridges a 2' shoulder is acceptable on a low speed, low volume roadway having a good accident record. After rehabilitation work is completed, the bridge should not be Structurally Deficient or Functionally Obsolete. A sufficiency number greater than 80 is also required unless it is waived for safety and public interest.

WisDOT policy item:

Rate the bridge using LFR provided it was designed using ASD or LFD. There are instances, however, where the LRFR rating of an existing bridge is beneficial (e.g. There is no M/M_u reduction to shear capacity using LRFR, which can affect longer-span steel bridges). Please contact the BOS Development Section if using LRFR to rate bridges designed using ASD or LFD. Bridges designed using LRFD must be rated using LRFR.

Deck removal on prestressed girders is a concern as the contractors tend to use jackhammers to remove the deck. Contractors have damaged the top girder flanges using this process either by using too large a hammer or carelessness. With the wide-flange sections this concern is amplified. It is therefore suggested that the contractor saw cut the slab to be removed longitudinally close to the shear connectors. With the previously applied bond breaker, the slab should break free and then the contractor can clear the concrete around the shear connectors. Saw cutting needs to be closely monitored as contractors have sawn through steel girder flanges by not watching their blade depth.

In the rehabilitation or widening of bridge decks, it is often necessary to place concrete while traffic uses adjacent lanes. There has been considerable concern over the effects of traffic-induced vibrations on the quality of the concrete and its bond to existing concrete and its embedded reinforcing steel. Wisconsin bridge construction experience indicates that there are many cases where problems have occurred during deck pours with adjacent traffic. Consideration should be given to prohibiting traffic during the deck pour until concrete has taken its initial set.

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

40.5 Deck Overlays

If the bridge is a candidate for replacement or a new deck, serviceability may be extended 3 to 7 years by patching and/or overlaying the deck with only a 1 1/2" minimum thickness asphaltic mat on lightly traveled roadways. Experience indicates the asphalt tends to slow down the rate of deterioration while providing a smooth riding surface. However, these decks must be watched closely for shear or punching shear failures as the deck surface problems are concealed.

For applications where the deck is structurally sound and service life is to be extended there are other methods to use. A polymer modified asphaltic overlay may be used to increase deck service life by approximately 15 years. If the concrete deck remains structurally sound, it may be practical to remove the existing overlay and place a new overlay before replacing the deck.

A 1 1/2" concrete overlay is expected to extend the service life of a bridge deck for 15 to 20 years. On delaminated but structurally sound decks a concrete overlay is often the only alternative to deck replacement. Prior to placing the concrete overlay, a minimum of 1" of existing deck surface should be removed. On all bridges low slump Grade E concrete is the specified standard with close inspection of concrete consolidation and curing. If the concrete deck remains structurally sound; it may be practical to remove the existing overlay and place a second deck overlay before replacing the entire deck. After the concrete overlay is placed, it is very important to seal all the deck cracks. Experience shows that salt water passes thru these cracks and causes deterioration of the underlying deck.

On deck overlays preparation of the deck is an important issue after removal of the top surface. Check the latest Special Provisions and/or specifications for the method of payment for Deck Preparation where there are asphalt patches or unsound concrete.

Micro-silica concretes have been effectively used as an alternate type of concrete overlay. It provides excellent resistance to chloride penetration due to its low permeability. Micro-silica modified concrete overlays appear very promising; however, they are still under experimental evaluation. Latex overlays when used in Wisconsin have higher costs without noticeable improved performance.

Ready mixed Grade E concrete with superplasticizer and fiber mesh have been tried and do not perform any better than site mixed concrete produced in a truck mounted mobile mixer.

Bridges with Inventory Ratings less than HS10 with an overlay shall not be considered for concrete overlays, unless approved by Structures Design. Bridges reconstructed with overlays shall have their new inventory and operating ratings shown on the bridge rehabilitation plans. Verify the desired transverse cross slope with the Regions as they may want to use current standards.

40.5.1 Guidelines for Bridge Deck Overlays

As a structure ages, rehabilitation is a necessary part of insuring a level of acceptable serviceability. Overlays can be used to extend the service lives of bridge decks that have surface deficiencies. Guidelines for determining if an overlay should be used are:

- The structure is capable of carrying the overlay deadload;
- The deck and superstructure are structurally sound;
- The desired service life can be achieved with the considered overlay and existing structure;
- The selected option is cost effective based on the structure life.

40.5.2 Deck Overlay Methods

An AC Overlay or Polymer Modified Asphaltic Overlay should not be considered on a bridge deck which has a longitudinal grade in excess of four percent or an extensive amount of stopping and starting traffic. All full depth repairs shall be made with PC concrete.

Guidelines for determining the type of deck overlay method to achieve the desired extended service life are:

AC Overlay (ACO): 5 years average life expectancy

- The minimum asphaltic overlay thickness is 1 1/2".
- The grade change due to overlay thickness can be accommodated at minimal cost.
- Deck or bridge replacement is programmed within 7 years.
- Raising of floor drains or joints is not required.
- Spalls can be patched with AC or PC concrete with minimal surface preparation.

Polymer Modified Asphaltic Overlay: 15 to 20 years life expectancy

- This product may be used as an experimental alternate to LSCO given below. CAUTION – Core tests have shown the permeability of this product is dependent on the aggregate. Limestone should not be used.

AC Overlay with a Waterproofing Membrane (ACOWM): (Currently not used)

Low Slump Concrete Overlay *(LSCO): 15 to 20 years life expectancy

- Minimum thickness is 1 1/2" PC concrete overlay.
- Joints and floor drains will be modified to accommodate the overlay.
- Deck deficiencies will be corrected with PC concrete.
- The prepared deck surfaces will be scarified or shot blasted.
- There is no structural concern for excessive leaching at working cracks.

- Combined distress area is less than 25%.

- May require crack sealing the following year and periodically thereafter.

* Note: Or another PC concrete product as approved by Structures Development and coordinated with the Region.

40.5.3 Maintenance Notes

- All concrete overlays crack immediately. If the cracks in the deck are not sealed periodically, the rate of deterioration can increase rapidly.

- AC overlays with a waterproofing membrane can also be used on new decks or older decks that are in good condition as preventive maintenance.

40.5.4 Special Considerations

On continuous concrete slab bridges with extensive spalling in the negative moment area, not more than 1/3 of the top bar steel should be exposed if the bar ends are not anchored. This is to maintain the continuity of the continuous spans and should be stated on the final structure plans.

If more than 1/3 of the steel is exposed, either the centers of adjacent spans must be shored or only longitudinally overlay 1/3 of the bridge at a time.

40.5.5 Railings and Parapets

The top of the overlay should not go above the 3-inch vertical portion of a concrete parapet. Additionally, overlays increase vehicle lean over sloped face parapets resulting in vehicles on bridges with higher ADT and/or speed having an increased likelihood of impact with lights/obstructions on top of, or behind, the parapet.

Sub-standard railings and parapets should be improved. An example of such a sub-standard barrier would be a curb with a railing or parapet on top. Contact the Bureau of Structures Development section to discuss solutions.

40.6 Deck Replacements

Depending on the structure age or site conditions, the condition of original deck or deck overlay, a complete deck replacement may be the most cost effective and extend the life of the bridge by 40 years or more. Epoxy coated rebars are required on bridge deck replacements under the same criteria as for new bridges. The new deck and parapet or railing shall be designed per the most recent edition of the WisDOT Bridge Manual, including continuity bars and overhang steel. Refer to the criteria in 40.3 of this chapter for additional 3R project considerations. The top flange of steel girders should be painted.

The following condition or rating criteria are the minimum requirements on STN bridges eligible for deck replacements:

Item	Existing Condition	Condition after Construction
Deck Condition	≤ 4	≥ 8
Inventory Rating	≥ HS15	≥ HS15
Superstructure Condition	≥ 3	≥ 8
Substructure Condition	≥ 3	≥ 8
Horizontal and Vertical Alignment Condition	> 3	---
Shoulder Width	6 ft	6 ft

Table 40.6-1
Condition Requirements for Deck Replacements

When the structure is a continuous steel girder bridge and meets criteria for deck replacement, but has an Inventory Rating less than HS10, a bridge replacement is recommended. On all Interstate Highway deck replacements, the bridges are to have a minimum Inventory Rating of HS20, after the deck is replaced.

For slab superstructure replacements on concrete pier columns, it is reported that the columns often get displaced and/or cracked during removal of the existing slab. Plans should be detailed showing full removal of these pier columns to the existing footing and replacement with current standard details showing a concrete cap with pier columns or shaft.

See the *Facilities Development Manual* for anchorage/offset requirements for temporary barrier used in staged construction.

In general, the substructure need not be analyzed for additional dead load provided the new deck is comparable to the existing. Exceptions include additional sidewalk or raised medians that would significantly alter the dead load of the superstructure.

For prestressed girder deck replacements, only use intermediate steel diaphragms in locations of removed intermediate concrete diaphragms (i.e. don’t add intermediate lines of diaphragms).

40.7 Rehabilitation Girder Sections

WisDOT BOS has retired several girder shapes from standard use on new structures. The 45”, 54” and 70” girder sections shall be used primarily for bridge rehabilitation projects such as girder replacements or widening.

These sections may also be used on a limited basis on new curved structures when the overhang requirements cannot be met with the wide-flange girder sections. These sections employ draped strand patterns with undraped alternates where feasible. Undraped strand patterns, when practical, should be specified on the designs. See the Standard Details for the girder sections’ draped and undraped strand patterns.

The 45”, 54”, and 70” girders in Chapter 40 standards have been updated to include the proper non-prestressed reinforcement so that these sections are LRFD compliant. Table 40.7-1 provides span lengths versus interior girder spacings for HL-93 live loading on single-span and multiple-span structures for prestressed I-girder rehabilitation sections. Girder spacing and span lengths are based on the following criteria:

- Interior girders with low relaxation strands at $0.75f_{pu}$,
- A concrete haunch of 2 1/2",
- Slab thicknesses from Chapter 17 – Superstructures - General
- A future wearing surface of 20 psf.
- A line load of 0.300 klf is applied to the girder to account for superimposed dead loads.
- 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 = 2” or 2 ½”
- Required f'_c girder at initial prestress < 6,800 psi

45" Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	102	112
6’-6”	100	110
7’-0”	98	108
7’-6”	96	102
8’-0”	94	100
8’-6”	88	98
9’-0”	88	96
9’-6”	84	90
10’-0”	84	88
10’-6”	82	86
11’-0”	78	85
11’-6”	76	84
12’-0”	70	80

54" Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	130	138
6’-6”	128	134
7’-0”	124	132
7’-6”	122	130
8’-0”	120	128
8’-6”	116	124
9’-0”	112	122
9’-6”	110	118
10’-0”	108	116
10’-6”	106	112
11’-0”	102	110
11’-6”	100	108
12’-0”	98	104

70” Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	150*	160*
6’-6”	146*	156*
7’-0”	144*	152*
7’-6”	140*	150*
8’-0”	138*	146*
8’-6”	134*	142*
9’-0”	132*	140*
9’-6”	128*	136
10’-0”	126*	134
10’-6”	122	132
11’-0”	118	128
11’-6”	116	126
12’-0”	114	122

Table 40.7-1
Maximum Span Length vs. Girder Spacing

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

40.8 Widenings

Deck widenings, except on the Interstate, are attached to the existing decks if they are structurally sound and the remaining width is more than 50 percent of the total new width. Also, reference is made to the criteria in 40.3 of this Chapter for additional 3R project considerations. If the existing deck is over 20 percent surface delaminated or spalled, the existing deck shall be replaced. For all deck widenings on Interstate Highway bridges, the total deck should be replaced in order that total deck life is equal and costs are likely to be less when considering future traffic control. Evaluate the cost of traffic control for deck widenings on other highway bridges. The total deck should be replaced in these cases where the life-cycle cost difference is minimal if future maintenance costs are substantially reduced.

The design details must provide a means of moment and shear transfer through the joint between the new and existing portions of the deck. Lapped reinforcing bars shall have adequate development length and are preferable to doweled bars. The reinforcing laps must be securely tied or the bars joined by mechanical methods. When practical, detail lapped rebar splices. Mechanical splice couplers or threaded rebar couples are expensive and should only be detailed when required. Generally, shear transfer is more than sufficient without a keyway. Bridge Maintenance Engineers have observed that if the existing bar steel is uncoated, newly lapped coated bars will accelerate the uncoated bar steel deterioration rate.

When widening a deck on a prestressed girder bridge, if practical, use the latest standard shape (e.g. 54W" rather than 54"). Spacing the new girder(s) to maintain comparable bridge stiffness is desirable. *The girders used for widenings may be the latest Chapter 19 sections designed to LRFD or the sections from Chapter 40 designed LFD, or LRFD with non-prestressed reinforcement as detailed in the Standard Details.*

For multi-columned piers, consider the cap connection to the existing cap as pinned and design the new portion to the latest LRFD criteria. The new column(s) are not required to meet AASHTO 3.6.5 (400 kip loading) as a widening is considered rehabilitation. *It is intended to provide standard details in the Bridge Manual for a crash barrier that could, at the option of the Region, be used to strengthen and provide motorists protection for existing piers, including widenings.*

Abutments shall be widened to current LRFD criteria as well as the Standard for Abutment Widening.

For foundation support, use the most current method available.

All elements of the widening shall be designed to current LRFD criteria. Railings and parapets placed on the new widened section shall be up to current design and safety standards. For the non-widened side, substandard railings and parapets should be improved. Contact the Bureau of Structures Development section to discuss solutions.

For prestressed girder widenings, only use intermediate steel diaphragms in-line with existing intermediate diaphragms (i.e. don't add intermediate lines of diaphragms).

40.9 Superstructure Replacements/Moved Girders (with Widening)

When steel girder bridges have girder spacings of 3' or less and require expensive redecking or deck widening; consider replacing the superstructure with a concrete slab. Approval is required from BOS for all Superstructure replacement projects. Also, additional costs will occur if bearings, floor drains, expansion joints, railings, or other structural elements require replacement.

When replacing a deck on a steel girder bridge, the cost of painting and structural rehab shall be considered to determine whether a deck or superstructure replacement is more cost effective (provided the substructure is sufficient to support the loading).

Evaluate the existing piers using current LRFD criteria. If an existing multi-columned pier has 3 or more columns, the 400 kip vehicular impact loading need not be considered if the pier is adjacent to a roadway with a design speed ≤ 40 mph. If the design speed is 45 mph or 50 mph, the 400 kip vehicular impact loading need not be considered if a minimum of "vehicle protection" is provided as per FDM 11-35-1. For design speeds > 50 mph, all criteria as per 13.4.10 must be met.

For abutments, evaluate the piles, or bearing capacity of the ground if on spread footings, utilizing Service I loading. The abutment body should be evaluated using Strength I loading.

The superstructure shall be design to current LRFD criteria.

40.16 Concrete Masonry Anchors for Rehabilitation

“Type S” and “Type L” concrete masonry anchors are used mostly for bridge rehabilitation projects and anchoring rail posts. One of the main differences between the two types of anchors is the duration of loading. It may be helpful to think of the “S” as Short-term loading and the “L” as Long-term loading. “Type L” anchors have greater embedment lengths to account for longer duration loading.

For both types of anchors the minimum pullout capacity specified on the plan is equal to (A_s x f_y), where f_y equals 60 ksi for rebar and 42 ksi for stainless steel bolts. The nominal resistance values shown in Table 40.16-1 and Table 40.16-2 are used in conjunction with the resistance factor, ϕ, for both tension and shear equal to 0.65. (*AASHTO* currently does not have resistance factors for anchors.) *It should be noted that WisDOT is currently evaluating adhesive anchored parapet replacements, which would be crash tested and consideration of the resistance factor would not be required*.

For all non-mechanical anchors, a two-part adhesive is either mixed and poured into a drilled hole or pumped into the hole by a dispenser which combines the two components at the nozzle just prior to entering the hole or within the hole. The hole must be properly cleaned and a sufficient amount of adhesive used so that the hole is completely filled with adhesive when the rebar or bolt is inserted.

"Type S" anchors are either mechanical wedge or adhesive anchors for installing studs, rebar, or bolts of a designated size. They are primarily used for anchoring bolts for attaching rail posts or other bolted objects and mostly smaller size rebars. “Type S” mechanical wedge anchors are seldom used for bridge rehabilitation. Because of creep, shrinkage and deterioration under load and freeze-thaw cycles, "Type S" adhesive anchors should not be used in situations where the rebar experiences a constant tension stress. When “Type S” anchors are used to anchor rebars, the rebar is listed in the “Bill of Bars” and paid for under the bid item “Bar Steel Reinforcement HS (Coated) Bridges”.

“Type L” anchors are adhesive anchors used to anchor rebars when the rebar is subject to continuous loading. “Type L” anchors of adequate length are capable of developing the tension strength of the bar for indefinite periods of time. When embedded a development length, rebars will develop the nominal tensile resistance value for sustained loading with a “Type L” anchor. “Type L” anchors are typically used for abutment and pier widenings, but may be used in other applications. For “Type L” anchors the rebar is listed in the “Bill of Bars” and paid for under the bid item “Bar Steel Reinforcement HS (Coated) Bridges”.

Usage Restrictions: 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. **Contact the Bureau of Structures if considering any extension of a hammerhead pier (without additional vertical support from an added column) or for vertical overhead installations.**

The manufacturer and product name of the “Type L” anchors and “Type S” adhesive anchors used by the contractor must be on the Department’s approved product list for “Concrete Masonry Anchors, Type L”.

Type S Anchor Size	Embedment Depth* in	Minimum Spacing in	Minimum Edge Distance in	Nominal Tensile Resistance kips	Resist. Factor ϕ	Factored Tensile Resistance kips
#4 or 1/2”	5	8	4	12	0.65	8
#5 or 5/8”	6	8	4	19	0.65	12
#6 or 3/4”	7	8	4	26	0.65	17
#7 or 7/8”	7	12	6	36	0.65	23
#8 or 1”	9	12	6	47	0.65	30
#9 or 1-1/8”	11	12	6	60	0.65	39

Table 40.16-1

Design Table for Concrete Masonry Anchors, Type S

(* Embedment for adhesive anchors, all bars. Mechanical anchor embedment by manufacturer. For anchors with f_y not equal to 60 ksi, adjust resistance accordingly)

Type L Anchor Size	Coated Rebar Embedment Depth ft-in	Uncoated Rebar Embedment Depth ft-in	Minimum Spacing in	Minimum Edge Distance in	Nominal Tensile Resistance kips	Resist. Factor ϕ	Factored Tensile Resistance kips
#4	1-0	1-0	8	4	12	0.65	8
#5	1-6	1-0	8	4	19	0.65	12
#6	1-10	1-3	8	4	26	0.65	17
#7	2-5	1-8	12	6	36	0.65	23
#8	3-3	2-2	12	6	47	0.65	30
#9	4-1	2-9	12	6	60	0.65	39

Table 40.16-2

Design Table for Concrete Masonry Anchors, Type L

The Factored Nominal Shear Resistance value equals the Factored Tensile Resistance multiplied by 0.66. This value is for 3.0 ksi concrete and above. If used in lower strength concrete, reduce the value by the ratios of the concrete moduli of rupture.

The Nominal and Factored Nominal Tensile Resistance values in the tables vary linearly with Embedment Depth. Use at least 80% of the Embedment Depths specified in the tables for Type S or Type L anchors.

The required minimum pullout capacity (as stated on the plans) is equal to the Nominal Tensile Resistance.

Typical notes for bridge plans (shown in all capital letters):

MASONRY ANCHORS TYPE S 5/8-INCH. MIN. PULLOUT CAPACITY OF 19 KIPS. FOR ADHESIVE ANCHORS, EMBED A MINIMUM OF 6" IN CONCRETE.

When using "Type S" anchors to anchor bolts or studs, the bolt, nut, and washer or the stud as detailed on the plans is included in the bid item.

For "Type S" anchors using rebar, the rebar is listed in the "Bill of Bars" and paid for under the bid item "Bar Steel Reinforcement HS (Coated) Bridges".

MASONRY ANCHORS TYPE L NO. 5 BARS. MIN. PULLOUT CAPACITY OF 19 KIPS. EMBED A MINIMUM OF 1'-0" IN CONCRETE.

For "Type L" anchors, the rebar is listed in the "Bill of Bars" and paid for under the bid item "Bar Steel Reinforcement HS (Coated) Bridges".

It should be noted that AASHTO is considering adding specifications pertaining to concrete masonry anchors. This chapter will be updated once that information is available.

40.17 Plan Details

1. Excavation for Structures on Overlays

   There is considerable confusion on when to use or not use the bid item "Excavation for Structures" on overlay projects. In order to remove the confusion the following note is to be added to all overlay projects that only involve removal of the paving block or less.

   Any excavation required to complete the overlay or paving block at the abutments is to be considered incidental to the bid item "Concrete Masonry Overlay Decks".

   If the overlay project has any excavation other than the need to replace a paving block or prepare the end of the deck for the overlay the "Excavation" bid item should be used and the above note left off the plan.

2. For steel girder bridge deck replacements; show the existing exterior girder size including the lower flange size on the plans to assist the contractor in ordering falsework brackets.

3. On structure deck overlay projects:

   Verify desired transverse or roadway cross slopes with Regions considering the changes in bridge rating and approach pavement grade. Although relatively flat by today's standard of an 0.02'/' cross slope; a cross slope of 0.01'/'/0.015'/' may be the most desirable.

   The designer should evaluate 3 types of repairs. (Preparation Decks Type 1) is concrete removal to the top of the bar steel. (Preparation Decks Type 2) is concrete removal below the bar steel. (Full Depth Deck Repair) is full depth concrete removal and repair. The designer should compute the quantity and cost for each type and a total cost. Compare the total cost to the estimated cost of a deck replacement and proceed accordingly. Show the location of (Full Depth Deck Repair) on the plan sheet.

   Contractors have gotten projects where full depth concrete removal was excessive and no locations designated. It would have been a better decision to replace the deck.

4. When detailing two stage concrete deck construction; consider providing pier cap construction joints to coincide with the longitudinal deck construction joint. Also, transverse deck bar steel lap splicing is preferred over the use of bar couplers. This is applicable where there is extra bridge width and the falsework can be left for both deck pours.

5. Total Estimated Quantities

   For all Bituminous Material Overlays:

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45.1 General

The 1967 collapse of the Silver Bridge in West Virginia prompted the development of the National Bridge Inspection Standards (NBIS) which require each State Highway Department of Transportation to inspect, prepare reports, and determine load ratings for bridge structures on all public roads. Soon after the development of the NBIS, supporting documents, including the FHWA Bridge *Inspector's Reference Manual* and the AASHTO *Manual for Condition Evaluation of Bridges* were developed to help in implementing these standards.

In 2003, the AASHTO *Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges (AASHTO LRFR) was rel*eased. The manual represents a major overhaul of the earlier *Manual for Condition Evaluation of Bridges*. Although the manual emphasizes the LRFR method, it also provides rating procedures for the Load Factor Rating (LFR) methodology. For this reason, it will be the governing manual utilized by WisDOT for load rating structures.

Bridge load ratings are performed for specific purposes, such as: National Bridge Inventory (NBI) reporting, overweight permit load checks, bridge rehabilitation, etc. However, the main purpose of load rating is to determine the safe live load capacity of a structure. It would be convenient if some simple measure such as gross vehicle weight could be used to determine a bridge's capacity. However, the actual capacity depends on many factors, such as the gross vehicle weight, the axle configuration, the distribution of loads between the axles, etc. It is generally accepted that a bridge that can carry a given load on two axles can carry the same load or a larger load spread over several axles. Since it is not practical to rate a bridge for the nearly infinite number of axle configurations of trucks on our highways, bridges are rated for standard vehicles which are representative of the actual vehicles in use today. These standard vehicles will be discussed later in this document.

Whenever a bridge on the State Trunk Highway System is not able to safely carry the loads allowed by State Statute, it is load posted. Current Wisconsin State Statutes allow a gross vehicle weight of 80,000 pounds while loads up to 170,000 pounds are allowed for annual permit loads.

The FHWA currently requires that two capacity ratings, referred to as the Inventory Rating and Operating Rating be submitted with the NBI file. The Inventory Rating is the load level that a structure can safely sustain for an indefinite period. The Operating Rating is the absolute maximum permissible load level to which a structure may be subjected. The FHWA requires that the standard AASHTO HS truck or lane loading be used as the vehicle when load rating with the Load Factor Rating method (LFR) and that the AASHTO HL-93 loading be utilized as the vehicle when load rating with the Load and Resistance Factor method (LRFR). A detailed explanation of each method, as well as a guide for when to utilize each method can be found in 45.3.

Bridges being analyzed for staged construction, whether new or rehabilitation, shall satisfy the requirements of LRFR (or LFR, 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.

45.2 Bridge Inspections

To determine the strength or load carrying capacity of a bridge, it is necessary to have a complete description of the bridge. This can include as-built plans, repair records, photographs, design/rating calculations, and current inspection information. If drawings are not available or are incomplete, they must be reproduced by means of complete measurements taken in the field. The present condition can be gathered from recent field inspection reports.

Inspections of bridges on the State Trunk Highway Network are performed by trained personnel from the Regional Maintenance Sections utilizing guidelines established in the latest edition of the *WisDOT Structure Inspection Manual*. Engineers from the Bureau of Structures may assist in the inspection of bridges with unique structural problems or when it is suspected that a reduction in load capacity is warranted. To comply with the National Bridge Inspection Standards (NBIS), it is required that all bridges on Federal Aid Routes be routinely inspected at intervals not to exceed two years. More frequent inspections are performed for bridges which are posted for load capacity or when it is warranted by their condition. In addition, special inspections such as underwater diving or fracture critical are performed when applicable.

Inspectors enter inspection information into the Highway Structures Information System (HSI), an online Bridge Management System developed by WisDOT. This database is used to create the NBI file and is also the central source for documents such as plans, maintenance records, design calculations and rating calculations that are critical when calculating structural ratings.

HSI also supplies a “re-rate flag” on the bridge inspection form that allows the inspector to schedule a structure for rating analysis if field conditions dictate the need. This flag can be queried by owners to obtain a quick list of structures needing analysis at the end of the inspection cycle. Ratings for State Owned Structures are generally performed by Bureau of Structures staff. Load Ratings for Local Owners are the responsibility of the owner.

45.2.1 Condition of Bridge Members

The condition and extent of deterioration of structural components of the bridge should be considered in the computation of the dead load and live load effects for the capacity when shear or moment is chosen for use in the basic rating equation.

As mentioned above, the rating of an older bridge for its load-carrying capacity should be based on a current field inspection. All physical features of a bridge which have an effect on the structural integrity should be examined. The inspector should note any damaged or deteriorated sections and obtain adequate data on these areas so that their effect can be properly evaluated in the analysis. Where steel is severely corroded, concrete deteriorated, or timber decayed, the inspector should make a determination of the loss in a cross-sectional area as closely as reasonably possible. They should also determine if deep pits, nicks or other defects exist that may cause stress concentration areas in any structural member. Lowering load capacities may be necessary if such conditions exist.

If not otherwise noted in the plans, inspectors should note the size, number, and the relative location of bolts and rivets through tension members so that the net area of the section can be calculated. Any misalignment, bends, or kinks in compression members should also be measured carefully. Such defects will have a great effect on the load-carrying capability of a member and may be the controlling factor in the load-carrying capacity of the entire structure. Also, the inspector should examine the connections of compression members carefully to see if they are detailed such that eccentricities are introduced which must be considered in the structural analysis.

The effective area of members to be used in the calculations shall be the gross area less that portion which has deteriorated due to decay or corrosion. The effective area should be adjusted for rivet or bolt holes in accordance with the AASHTO LRFD Design Specifications or the AASHTO Standard Specifications, where applicable.

45.3 Load Rating Methodologies

There are two primary methods of load rating bridge structures that will be utilized by WisDOT. Both methods are detailed in the AASHTO LRFR manual. They are as follows:

- Load Factor Rating (LFR)
- Load and Resistance Factor Rating (LRFR)

LFR has been used since the early 1990's to load rate bridges in Wisconsin. The basic philosophy behind this method is to assign factors of safety to both dead and live loads. Loads that are more predictable, such as dead loads, are assigned a lower factor of safety while loads that are less predictable, such as truck loads, are assigned a higher factor of safety. The rating is determined such that the effect of the factored loads does not exceed the strength of the member. LFR will be utilized on all structures in the WisDOT inventory designed with either LFD or ASD and will be thoroughly covered in 45.3.3. A detailed description of this method can also be found in **LRFR [D.6.1]**.

LRFR employs the same basic principles as LFR for the load factors, but also utilizes resistance factors to account for uncertainties in member condition, material properties, etc. This methodology will be used for all structures designed using Load and Resistance Factor Design (LRFD). This method will be covered in 45.3.2. A detailed description of this method can be found in **LRFR [6.1]**.

WisDOT policy item:

Rate the bridge using LFR provided it was designed using ASD or LFD. There are instances, however, where the LRFR rating of an existing bridge is beneficial (e.g. There is no M/M_u reduction to shear capacity using LRFR, which can affect longer-span steel bridges). Please contact the BOS Development Section if using LRFR to rate bridges designed using ASD or LFD. Bridges designed using LRFD must be rated using LRFR.

45.3.1 General Assumptions

The following concepts shall be applied to the load rating of structures in Wisconsin:

- The AASHTO *LRFR* manual has provisions for both LRFR and for LFR and shall be the main load rating manual for WisDOT. This chapter serves as a supplement to the LRFR manual and deals primarily with WisDOT exceptions, interpretations, and policy decisions.
- Substructures generally do not control the load rating. Therefore, a complete analysis of the substructure is not required if, in the judgment of the load rating engineer, the substructure has greater capacity than the superstructure.
- Reinforced concrete bridge decks on redundant, multi-girder bridges need not be rated unless damage, deterioration, or other concerns merit this analysis, as determined by the judgment of the load rating engineer.

- Dead loads shall be distributed as described in 17.2.7 for slabs and 17.2.8 for slab on girders.

- Live loads shall be distributed as described in 17.2.7 or 18.4.5.1 for slabs and 17.2.8 for slab on girders.

- 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.

- For continuous girder type bridges, the negative moment steel shall conservatively consist of only the top mat of steel over the piers.

- 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.

- The governing rating shall be the lesser of the shear capacity or moment capacity of the critical component. If the load rating engineer, utilizing engineering judgment, determines that certain components will not control the rating, then a full analysis of the non-controlling element is not required.

45.3.2 Load and Resistance Factor Rating (LRFR) Method

All bridge structures designed utilizing Load and Resistance Factor Design (LRFD) shall be rated LRFR per the 2003 AASHTO *Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges.* The basic rating equation, per **LRFR [Equation 6-1]**, is:

$$RF = \frac{C - \gamma_{DC} DC - \gamma_{DW} DW}{\gamma_L (LL + IM)}$$

For the Strength Limit States:

$$C = \phi_C \phi_S \phi R_n$$

Where the following lower limit shall apply:

$$\phi_C \phi_S \geq 0.85$$

Where:

RF	=	Rating Factor
C	=	Capacity

R_n	=	Nominal member resistance
DC	=	Dead-load effect due to structural components and attachments
DW	=	Dead-load effect due to the wearing surface and utilities
LL	=	Live load effects
IM	=	Dynamic load allowance
γ_{DC}	=	LRFR load factor for structural components and attachments
γ_{DW}	=	LRFR load factor for wearing surfaces and utilities
γ_L	=	LRFR load factor for live load
ϕ_C	=	Condition Factor
ϕ_S	=	System Factor
ϕ	=	LRFR resistance Factor

The LRFR methodology is comprised of three distinct procedures:

- Design Load Rating (first level evaluation)
- Legal Load Rating (second level evaluation)
- Permit Load Rating (third level evaluation)

The results of each procedure serve specific uses and also guide the need for further evaluations to verify bridge safety or serviceability. A flow chart outlining this approach is shown in Figure 45.3-1. The procedures are structured to be performed in a sequential manner, as needed, starting with the Design Load Rating. Load rating for AASHTO legal loads is only required when a bridge fails (RF<1) the Design Load Rating at the Operating level.

Note that when designing a new structure, it is required that the rating factor be greater than one for the HL-93 vehicle at the Inventory Level; therefore, a Legal Load Rating will never be required on a newly designed structure.

Similarly, only bridges that pass the Legal Load Rating (RF≥1) can be evaluated utilizing the Permit Load Rating procedures. This level is used for both the Wisconsin Standard Permit Vehicle Design Check, as discussed in 45.6 and for Single Trip permit evaluation as discussed in 45.7.3.

45.3.2.8 Permit Load Rating

This level of load rating serves many purposes for WisDOT. First, it is the level of load rating analysis required for all structures when performing the Wisconsin Standard Permit Vehicle Design Check as illustrated in 45.6. Second, this level is used, whenever necessary, for issuance of Single Trip permits. As their name indicates, single trip permits are valid for only one trip. Each single trip permit vehicle is analyzed for every structure it will cross.

45.3.2.8.1 Permit Load Rating Live Load

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed (Figure 45.6-1). Specifics on this analysis can be found in 45.6.

For specific Single Trip permit applications, the actual truck configuration described in the permit shall be the live load used to analyze all pertinent structures.

WisDOT policy items:

WisDOT interpretation of **LRFR [6.4.5.4.1]** is for spans up to 200'-0", only the permit vehicle shall be considered present in the lane. For spans 200'-0" in length or greater an additional lane load shall be applied to simulate closely following vehicles. The lane load shall be taken as 0.2 klf in each lane and shall only be applied to those portions of the span(s) where the loading effects add to the permit load effects.

Also note, as stated in the footnote of **LRFR [Table 6-6]**, when using a single-lane LRFD distribution factor, the 1.2 multiple presence factor should be divided out from the distribution factor equations.

45.3.3 Load Factor Rating (LFR) Method

All bridge structures designed utilizing LFD or ASD shall be rated (possible exception stated in Policy Item under 45.3) utilizing LFR per the 2003 AASHTO *Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* (**LRFR [Appendix D.6.1]**). The basic rating equation can be found in **LRFR [Equation D6.1]** and is:

$$RF = \frac{C - A_1 D}{A_2 (L + I)}$$

Where:

RF	=	Rating Factor
C	=	Capacity
D	=	The dead load effect on the member.

L = The live load effect on the member

I = The impact factor to be used with the live load effect

A_1 = Load Factor for Dead Load

A_2 = Load Factor for Live Load

45.3.3.1 Live Loads

Similar to LRFR, there are three potential checks to be made in LFR that are detailed in the flow chart shown in Figure 45.3-2. For purposes of calculating the Inventory and Operating rating of the structure, the live load to be used should be the HS20 truck or lane loading as shown in Figures 17.2-1 and 17.2-3. For conducting the Wisconsin Standard Permit Vehicle Design Check, use the loading shown in Figure 45.6-1. For determination of postings, refer to 45.4.1 for the proper posting vehicles.

One important item to note: when rating permit loads for continuous concrete slab bridges of 30'-0" width or more wheel loads are distributed over a width of 12'-0", which is a simplified adaptation of the distribution factor in the Ontario Bridge Design Code.

45.3.3.2 Load Factors

See Table 45.3-5 for load factors to be used when rating with the LFR method. The nominal capacity C is the same regardless of the rating level desired.

LFR Live Load Factors		
Rating Level	A_1	A_2
Inventory	1.3	2.17
Operating	1.3	1.3

Table 45.3-5
LFR Live Load Factors