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2.1 Organizational Charts

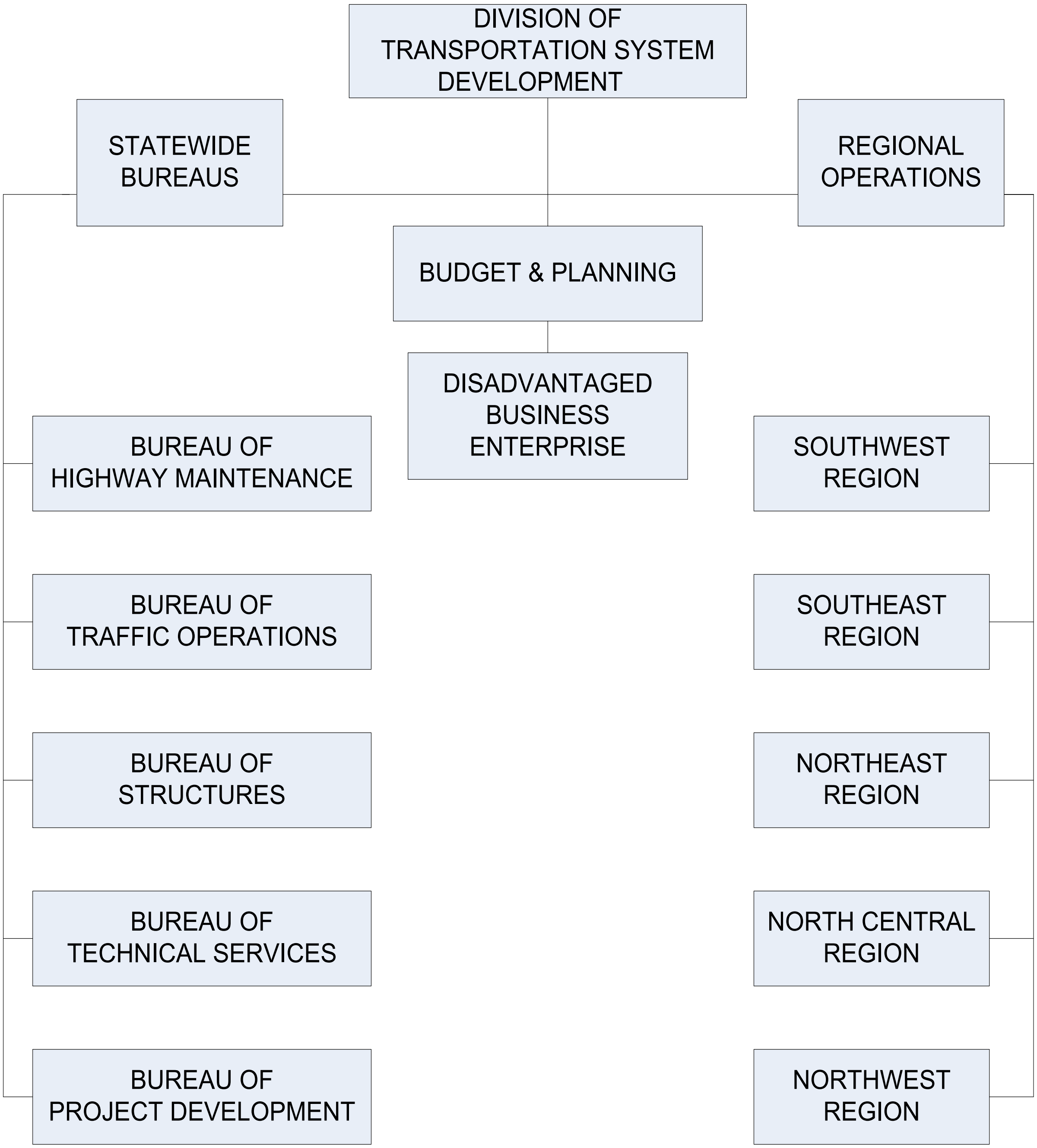


Figure 2.1-1
Division of Transportation System Development

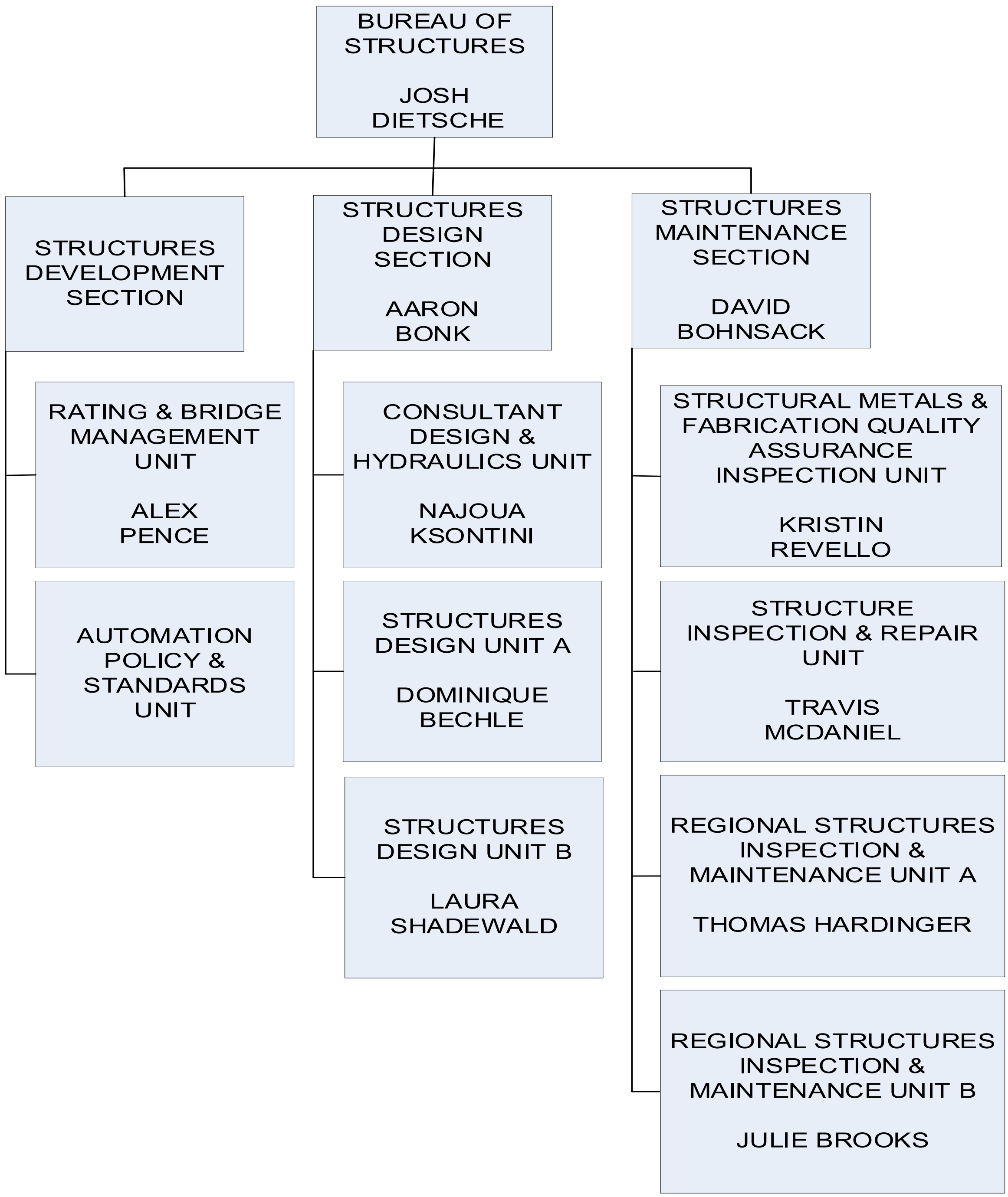
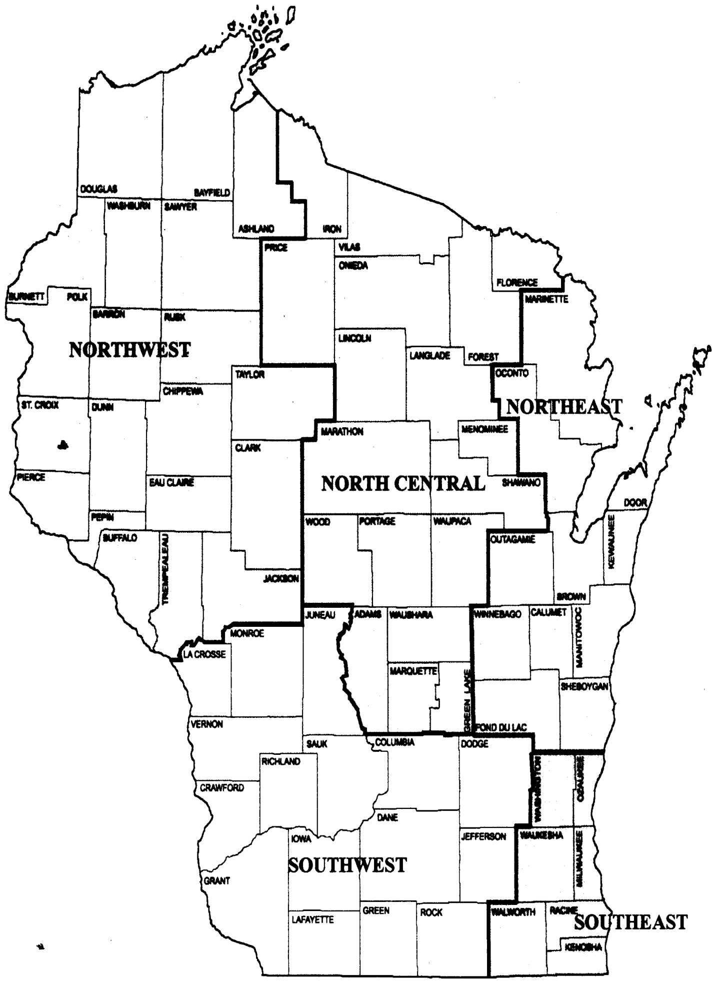


Figure 2.1-2
Bureau of Structures



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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6.3.3.3 Miscellaneous Structures

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

6.3.3.4 Standard Drawings

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

6.3.3.5 Insert Sheets

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

6.3.3.6 Change Orders and Maintenance Work

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

6.3.3.7 Name Plate and Benchmarks

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

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

6.3.3.8 Removing Structure and Debris Containment

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

The "Removing Structure (structure)" bid item is most typically used for complete or substantial removals, as described in 6.3.3.8.2, of grade separation structures. In addition to this Standard Specification bid item, there are three additional Standard Specification bid items for complete or substantial removal work over waterways: "Removing Structure Over Waterway Remove Debris (structure)"; "Removing Structure Over Waterway Minimal Debris (structure)"; and "Removing Structure Over Waterway Debris Capture (structure)". If these four Standard Specification bid items do not encapsulate site specific constraints for specialized cases, which should be a rare occurrence, the designer can utilize special provisions to augment the standard spec removal items.

The designer should review all of these Standard Specifications and coordinate with the Wisconsin Department of Natural Resources (DNR) to determine which bid items to use when removing a particular structure. **If the designer disagrees with the recommendation from the DNR's Initial Review Letter (IRL), the designer should work with WisDOT Regional Environmental Coordinator (REC), WisDOT Regional Stormwater & Erosion Control Engineer (SWECE) and DNR Transportation Liaison (TL) to come to a consensus on the appropriate bid item, considering constructability and cost impacts of the items.** For unique or difficult removals, designers should consult with the contracting community to assess costs and the feasibility of a particular removal technique. One of the following Removing Structure bid items should be selected for removals over waterways:

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

- Removing Structure Over Waterway Minimal Debris (structure) is used where it is possible to remove the structure with only minimal debris dropping into a waterway or wetland, and that waterway or wetland is not highly environmentally sensitive. This bid item is typically appropriate for removing all structure types except for the following bridges which are typically covered under Removing Structure Over Waterway Remove Debris (structure): slab spans; voided slabs; cast-in-place concrete girder bridges; earth-filled bridges; large trestle bridges. This bid item will likely be used for most stream crossing removals. The designer may need to expand the standard spec with special provision language to address additional DNR concerns and/or issues. CMM 645.6 contains example removal and clean-up methods corresponding to this bid item.

- Removing Structure Over Waterway Debris Capture (structure) is typically used when resources are present such that additional protection is required due to the waterway or wetland being highly environmentally sensitive. Before including this bid item in the contract, consult with the DNR and the department's regional environmental coordinator, as well as BOS, to determine if this bid item is appropriate. The designer

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12.6 Abutment Drainage and Backfill

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

12.6.1 Abutment Drainage

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

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

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

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

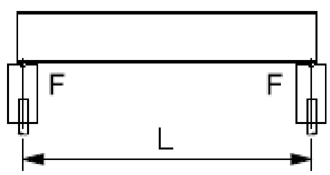
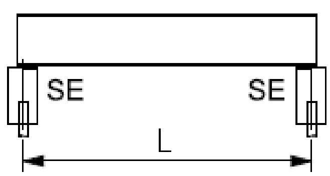
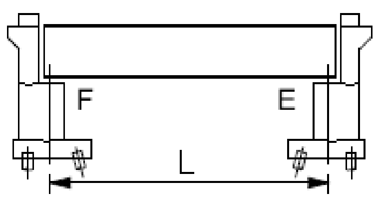
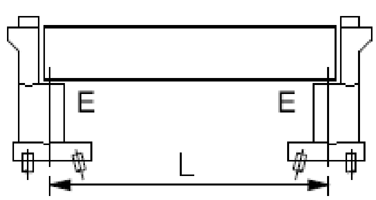
Pipe underdrains and weepholes may discharge water during freezing temperatures. In urban areas, this may create a problem due to the accumulation of flow and ice on sidewalks.

12.6.2 Abutment Backfill Material

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

12.7 Selection of Standard Abutment Types

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

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

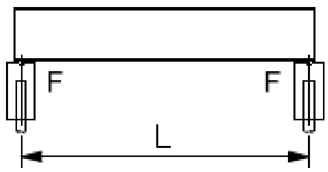
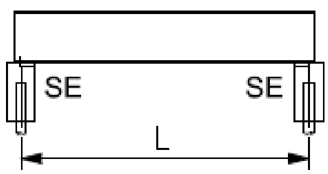
Abutment Arrangements	Superstructures		
	Concrete Slab Spans	Prestressed Girders	Steel Girders
Type A5 (F-F) 	$L \le 150'$ $S \le 30°$ $AL \le 50'$	$L \le 150'$ $S \le 15°$ $AL \le 50'$ 28" only (36W" thru 82W" require SE)	$L \le 150'$ $S \le 15°$ $AL \le 50'$
Type A5 (SE-SE) 	$L \le 200'$ $S \le 30°$ $AL > 50'$	$L \le 200'$ $S \le 30°$	$L \le 150'$ $S \le 30°$
ABUTMENT TYPES			

Figure 12.7-1
Recommended Guide for Abutment Type Selection

Where:

S = Skew

AL = Abutment Length

F = Fixed seat

SE = Semi-Expansion seat

E = Expansion seat

L = Length of continuous superstructure between abutments

Footnotes to Figure 12.7-1:

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

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

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36.1 Design Method

36.1.1 Design Requirements

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

36.1.2 Rating Requirements

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

36.1.3 Standard Permit Design Check

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

36.2 General

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

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

- Hydraulic and other requirements at the site determine the required height and area of the box. Hydraulic design of box culverts is described in Chapter 8.
- Once the required height and area is determined, the selection of a single or multi-cell box is determined entirely from economics. Barrel lengths are computed to the nearest 6 inches. For multi-cell culverts the cell widths are kept equal.
- A minimum vertical opening of 5 feet is desirable for cleaning purposes.

Pedestrian underpasses should consider the following items:

- The minimum opening for pedestrian underpasses is 8 feet high by 10 feet wide. However, when considering maintenance and emergency vehicles or bicyclists the minimum opening should be 10 feet high by 12 feet wide. For additional guidance refer to the Wisconsin Bicycle Facility Design Handbook and the FDM.
- The top and sides should be waterproofed for the entire length of the culvert.
- The top of the bottom slab should be sloped with a 1% normal crown to minimize moisture collecting on the travel path. Additionally, 0.5% to 1% longitudinal slope for drainage is recommended.
- Flared wings are recommended at openings. For long underpasses, lighting systems (recessed lights and skylights) should be considered, as well. For additional guidance on user’s comfort, safety measures, and lighting refer to the Wisconsin Bicycle Facility Design Handbook.

Cattle underpasses should consider the following items:

- The minimum size for cattle underpasses is 6 feet high by 5 feet wide.
- Consider providing a minimum longitudinal slope of 1%, desirable 3%, to allow for flushing, but not so steep that the stock will slip. Slopes steeper than 5% should be avoided.
- For additional guidance refer to the FDM.

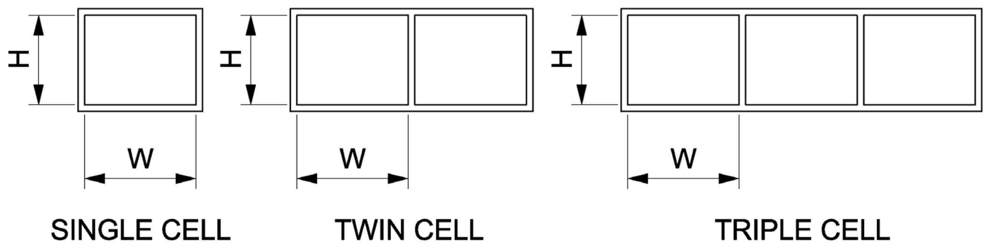

Figure 36.2-1
Typical Cross Sections

36.2.1 Material Properties

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

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

= 3.5 ksi for concrete in box culverts

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

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

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

Where:

K_1 = 1.0

W_C = 0.15 kcf, unit weight of concrete

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

36.2.2 Bridge or Culvert

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

36.3.3 Load Factors

In accordance with **LRFD [Table 3.4.1-1 and Table 3.4.1-2]**, the following Strength I load factors, γ_{st}, and Service I load factors, γ_{s1}, shall be used for box culvert design:

Type of Load		Strength I Load Factor, γ_{st} Max.	Min.	Service I Load Factor, γ_{s1}
Dead Load-Components	DC	1.25	0.90	1.0
Dead Load-Wearing Surface	DW	1.50	0.65	1.0
Vertical Earth Pressure	EV	1.30	0.90	1.0
Horizontal Earth Pressure	EH	1.35	0.50[1]	1.0
Live Load Surcharge	LS	1.75	1.75	1.0
Live Load + IM	LL+IM	1.75	1.75	1.0

[1]Per **LRFD [3.11.7]**, for culverts where earth pressure may reduce effects caused by other loads, a 50% reduction may be used, but not combined with the minimum load factor specified in **LRFD [Table 3.4.1-2]**.

36.3.4 Strength Limit State

Strength I Limit State shall be applied to ensure that strength and stability are provided to resist the significant load combinations that a structure is expected to experience during its design life **LRFD [1.3.2.4]**.

36.3.4.1 Factored Resistance

The resistance factor, ϕ, is used to reduce the computed nominal resistance of a structural element. This factor accounts for the variability of material properties, structural dimensions and workmanship, and uncertainty in prediction of resistance.

The resistance factors, ϕ, for reinforced concrete box culverts for the Strength Limit State per **LRFD [Table 12.5.5-1]** are as shown below:

Structure Type	Flexure	Shear
Cast-In-Place	0.90	0.85
Precast	1.00	0.90
Three-Sided	0.95	0.90

36.3.4.2 Moment Capacity

For rectangular sections, the nominal moment resistance, M_n, per **LRFD [5.6.3.2.3]** (tension reinforcement only) equals:

$$M_n = A_s f_s \left(d_s - \frac{a}{2}\right)$$

The factored resistance, M_r, or moment capacity per **LRFD [5.6.3.2.1]**, shall be taken as:

$$M_r = \phi M_n = \phi A_s f_s \left(d_s - \frac{a}{2}\right)$$

For additional information on concrete moment capacity, including stress and strain assumptions used, refer to 18.3.3.2.1.

The location of the design moment will consider the haunch dimensions in accordance with **LRFD [12.11.5.2]**. No portion of the haunch shall be considered in adding to the effective depth of the section.

36.3.4.3 Shear Capacity

Per **LRFD [12.11.5.1]**, shear in culverts shall be investigated in conformance with **LRFD [5.12.7.3]**. The location of the critical section for shear for culverts with haunches shall be determined in conformance with **LRFD [C5.12.8.6.1]** and shall be taken at a distance d_v from the end of the haunch.

36.3.4.3.1 Depth of Fill Greater than or Equal to 2.0 ft.

The shear resistance of the concrete, V_c, for slabs of box culverts with 2.0 feet or more of fill, for one-way action per **LRFD [5.12.7.3]** shall be determined as:

$$V_c = \left(0.0676\lambda\sqrt{f'_c} + 4.6\frac{A_s}{bd_e}\frac{V_u d_e}{M_u}\right)bd_e \leq 0.126\lambda\sqrt{f'_c}\,bd_e$$

Where:

$$\frac{V_u d_e}{M_u} \leq 1$$

Where:

V_c	=	Shear resistance of the concrete (kip)
A_s	=	Area of reinforcing steel in the design width (in^2)

36.4 Design Loads

36.4.1 Self-Weight (DC)

Include the structure self-weight based on a unit weight of concrete of 0.150 kcf. When there is no fill on the top slab of the culvert, the top slab thickness includes a ½" wearing surface. The weight of the wearing surface is included in the design, but its thickness is not included in the section properties of the top slab.

36.4.2 Future Wearing Surface (DW)

If the fill depth over the culvert is greater than zero, the weight of the future wearing surface shall be taken as zero. If there is no fill depth over the culvert, the weight of the future wearing surface shall be taken as 20 psf. This load is designated as, DW, dead load of wearing surfaces and utilities, for application of load factors and limit state combinations.

36.4.3 Vertical and Horizontal Earth Pressure (EH and EV)

The weight of soil above the buried structure is taken as 0.120 kcf. Use a 1.30 load factor for vertical earth pressure, in accordance with **LRFD [Table 3.4.1-2]** for rigid buried structures. A coefficient of lateral earth pressure of 0.5 is used for the lateral pressure from the soil. This coefficient of lateral earth pressure is based on an at-rest condition and an effective friction angle of 30º, **LRFD [3.11.5.2]**. The lateral earth pressure is calculated per **LRFD [3.11.5.1]**:

$$p = k_o \gamma_s z$$

Where:

p	=	Lateral earth pressure (ksf)
k_o	=	Coefficient of at-rest lateral earth pressure
γ_s	=	Unit weight of backfill (kcf)
z	=	Depth below the surface of earth fill or top of roadway pavement (ft)

WisDOT Policy Item:

For modification of earth loads for soil-structure interaction, embankment installations are always assumed for box culvert design, in accordance with **LRFD [12.11.2.2]**.

Soil-structure interaction for vertical earth loads is computed based on **LRFD [12.11.2.2]**. For embankment installations, the total unfactored earth load is:

$$W_E = F_e \gamma_s B_c H$$

In which:

$$F_e = 1 + 0.20\frac{H}{B_c}$$

Where:

W_E	=	Total unfactored earth load (kip/ft width)
F_e	=	Soil-structure interaction factor for embankment installations (F_e shall not exceed 1.15 for installations with compacted fill along the sides of the box section)
γ_s	=	Unit weight of backfill (kcf)
B_c	=	Outside width of culvert, as specified in Figure 36.4-1 (ft)
H	=	Depth of fill from top of culvert to surface of earth fill or top of roadway pavement (ft)

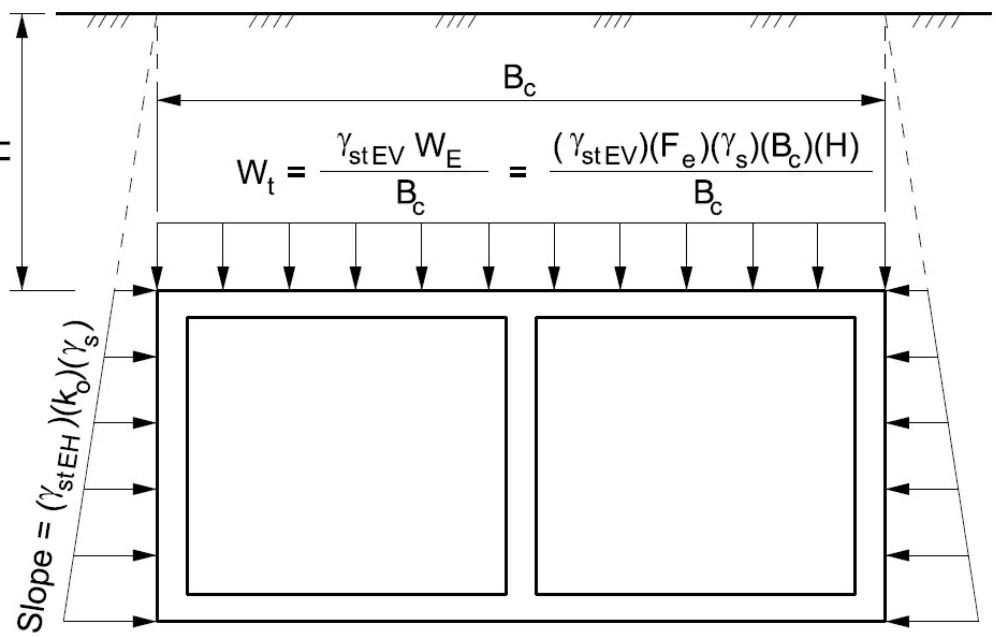

Figure 36.4-1
Factored Vertical and Horizontal Earth Pressures

Where:

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40.1 General

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

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

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

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

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

Cracks will develop in a new concrete deck throughout the first couple of years in response to vehicular and environmental loads. Initial concrete cracking should occur within the first two years of new deck construction. Placement after this time allows the overlay to seal existing cracks and may reduce reflective cracking in the overlay. Therefore, the earliest a thin polymer overlay shall be placed on a new deck is the following construction season. If it is determined that a thin polymer overlay should be placed in the next construction season, the thin polymer overlay should be included in the same contract as the new deck.
Thin polymer overlays can be used in lieu of resealing the deck on a project-to-project basis with BOS approval. Approval occurs through the structure certification process. Some examples where TPOs might be used instead of deck sealing are where heavy snowmobile traffic is expected or when the safety certification provides justification for enhanced friction surface treatment. See 40.5.5.1 for deck sealing usage in place of thin polymer overlays.

Sufficient bond strength is critical in maximizing the overlay's service life. The bond strength can be reduced by poor surface preparations, traffic conditions, moisture, and distressed concrete. As a result, TPO's should be used based on the following restrictions:

- Recommended on decks with a NBI rating greater than 7 to help mitigate chloride infiltration. The deck should be in good condition with wearing surface distressed areas not exceeding 2% of the total deck area.

- Not recommended on decks that have been exposed to chlorides for more than 10 years old or with a NBI rating less than 7. These restrictions assume that significant chloride infiltration has already occurred. When a robust deck washing and sealing program has been used, TPO's may be placed on decks 10-15 years old with above average deck condition.

- TPO's should not be placed on Portland cement concrete patches less than 28 days old. Patch and crack repairs shall be compatible with the overlay material.

- The earliest a thin polymer overlay shall be placed on a new deck is the following construction season. If it is determined through structure certification that a thin polymer overlay should be placed in the next construction season in lieu of future deck sealing, the thin polymer overlay should be included in the same contract as the new deck.

- Use of TPO's on the concrete approaches should be avoided. Slab-on-grade conditions may cause the overlay to fail prematurely due to moisture issues.

- Not recommended on decks with widespread cracking, large cracks (>0.04 in), or active cracks (e.g. longitudinal reflective cracks between PS box girders). These cracks are likely to reflect through the overlay, even when fully repaired.

- Decks with an existing TPO may be considered for a TPO re-application provided that the previously discussed restrictions can be assumed to be satisfied. Generally, this assumes the existing overlay performed well over its expected service life and the effective deck exposure did not exceed 15 years, such that significant chloride infiltration has not occurred. If signification chloride infiltration is expected, a re-application would not be recommended.

Thin polymer overlays may be considered where friction needs to be restored or improved. For deck applications, a two-layer polymer overlay system shall be used throughout the deck surface (driving lanes and shoulder) for deck preservation against chloride infiltration. Additionally, the two-layer application provides deck protection against snowplow and snowmobile operations. The “Polymer Overlay” bid item is the standard two-layer polymer overlay with natural or synthetic aggregates and provides improved or “enhanced” surface friction. For situations warranting a higher skid resistance, the bid item “High Friction Surface Treatment Polymer Overlay” with calcined bauxite aggregates shall be used. See Chapter 40 Standards and the Traffic Engineering, Operations & Safety Manual TEOpS 12-5-4 for additional guidance.

40.5.1.2 Low Slump Concrete Overlay

A low slump concrete overlay, also referred to as a concrete overlay, is expected to extend the service life of a bridge deck for 15 to 20 years. This system is comprised of low slump Grade E concrete and has a 1-1/2 inch minimum thickness. The overlay thickness can accommodate profile and cross-slope differences, but typically does not exceed 4-1/2 inches. Thicker overlays become increasingly unpractical due to load and cost implications.

Low slump Grade E concrete requires close adherence to the specification, including equipment, consolidation, and curing requirements. A properly cured concrete overlay will help limit cracks, but inevitably the concrete overlay will crack. After the concrete overlay has been placed, it is beneficial to seal cracks in the overlay to minimize deterioration of the underlying deck. The overlay may require crack sealing the following year and periodically thereafter.

On delaminated but structurally sound decks, a rehabilitation concrete overlay is often the only alternative to deck replacement. Typically, prior to placing the concrete overlay a minimum of 1” of existing deck surface is removed along with any unsound material and asphaltic patches.

Rehabilitation concrete overlays are performed when significant distress of the wearing surface has occurred. If more than 25% of the wearing surface is distressed, an in-depth cost analysis should be performed to determine if a concrete overlay is cost effective verses a deck replacement.

The quantity of distress on the underside of deck or slab should be negligible, less than 5%, indicating that the bottom mat of reinforcement steel is not significantly deteriorated. If significant quantities of distress are present under the deck, a deck replacement may be required in the future; an overlay at this time might not achieve full service life, but may be placed to provide a good riding surface until replacement.

If the structure has an existing overlay, the overlay condition should be evaluated in addition to the other previously discussed considerations. If the concrete deck remains structurally sound, it may be practical to remove an existing overlay and place a new overlay before replacing the entire deck. Prior to placing the concrete overlay, the existing overlay should be removed to at least the original deck surface. Additional surface milling may not be practical if the previous overlay included a milling operation.

40.5.1.3 Polyester Polymer Concrete Overlay

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

Prior to the placement of the PPC overlay, a high molecular weight methacrylate (HMWM) binder is placed on the prepared deck. This bonds the overlay to the deck, and it also serves to seal existing cracks in the deck. When the existing concrete is in good condition, PPC is effective at mitigating chloride penetration due to its impermeability.

The total thickness of a PPC overlay is typically 3/4” to 1”. While thicker overlays are possible, they are usually cost prohibitive. PPC can be placed at 3/4” thick as opposed to a typical 1 1/2” thick concrete overlay. This may help in situations where bridge ratings and/or profile adjustments are of concern.

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

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

- Decks should have a NBI rating of 7 or greater and be less than 15 years old. Older decks may be considered when the existing deck has been protected by a thin polymer overlay or when chloride testing indicates acceptable chloride levels at the reinforcement. Chloride contents at the reinforcement should not exceed 5 lbs/CY for decks with epoxy coated reinforcement. PPC overlays are not recommended on decks with uncoated top mat reinforcement. Decks exposed to chlorides, exceeding 10 years, should consider a ¾-inch minimum scarification to remove chlorides.

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

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

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

40.5.1.4 Polymer Modified Asphaltic Overlay

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

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

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

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

Note: PMA overlays are expensive, have a limited service life relative other overlay types, and product availability may be problematic. As a result, PMA overlays usage should be limited.

40.5.1.5 Asphaltic Overlay

An asphaltic overlay, without a waterproofing membrane, is expected to extend the service life of a bridge deck for 3 to 7 years. This system may be a viable treatment if the deck or bridge is programmed for replacement within 4 years on lightly traveled roadways and is able to provide a smooth riding surface. Without a waterproofing material, the overlay may trap moisture at the existing deck surface, which may accelerate deck deterioration.

These overlays must be watched closely for distress as the existing deck surface problems are concealed. This system is typically an asphaltic pavement with a mixture of aggregates and asphaltic materials, which can easily be placed as a deck overlay using conventional asphaltic mixing and placement equipment. The thickness of the overlay is 2-inches minimum and can accommodate profile and cross-slope differences.

Note: Asphaltic overlays, without a waterproofing membrane, are not eligible for federal funds.

40.5.1.6 Asphaltic Overlay with Waterproofing Membrane

An asphaltic overlay, with a waterproofing membrane, is currently being used on a very limited basis. This system is expected to extend the service life of a bridge deck for 5 to 15 years. Experience indicates that waterproofing membranes decrease the rate of deck deterioration by preventing or slowing the migration of water and chloride ions into the concrete.

In the 1990's, waterproofing membranes were actively used with asphaltic overlays for protecting existing decks, but were phased out by 2009 when they were restricted due to performance concerns and the inability to inspect the deck. As a result, low slump concrete or PMA overlays are currently recommended when deck or bridge replacements are programed beyond 4 years, unless approved otherwise.

Note: Asphaltic overlays, with a waterproofing membrane, requires prior-approval by the Bureau of Structures. This system is currently under review for possible improvements.

40.5.1.7 Other Overlays

Several other overlay systems have been used on past projects, but are generally not used currently. Use of these systems or other systems not previously mentioned require prior-approval by the Bureau of Structures.

- Micro-silica (silica-fume) modified concrete overlay – Provides good resistance to chloride penetration due to its low permeability.

- Latex modified concrete overlay – Provides a long-lasting overlay system with minimal traffic disruptions. Several other states are currently using this overlay method with hydrodemolition deck preparations.

- Reinforced concrete overlays:

    - Thin overlays (< 4 ½") – Uses a superplasticizer and fiber reinforcement (steel or synthetic) for additional crack control by reducing cracks and crack widths.

    - Thick overlays (≥ 4 ½") – Uses steel reinforcements, rebar or weld wire fabric, typically for new structural decks. This overlay is intended to provide at least one layer of steel reinforcement, in each direction, for crack control. This overlay is currently recommended for PS box girder superstructures, which allows for composite details and improved means to control longitudinal reflective cracking. For most cases, steel reinforcement is not required when rehabilitation overlays exceed 4 1/2 - inches. Use of low slump Grade E concrete may not be suitable when incorporating steel reinforcements.

40.5.2 Selection Considerations

The selection of an overlay type is made considering several factors to achieve the desired extended service life. Several of these factors are provided in Table 40.5-1 and Table 40.5-2 to aid in the selection of an overlay for the preservation and rehabilitation of decks.

Overlay Type	Thin Polymer Overlay	Low Slump Concrete Overlay	Polyester Polymer Concrete Overlay (2)	Polymer Modified Asphaltic Overlay	Asphaltic Overlay (4)	Asphaltic Overlay with Membrane (2)
Overlay Life Span (years)	7 to 15	15 to 20	20 to 30	10 to 15	3 to 7	5 to 15
Traffic Impact (6)	< 1 day	7 days +/-	< 1 day	1-2 days	1-2 days	1-2 days
Overlay Costs ($/SF) (1)	$3 to $5	$4 to $7	$8 to $18	$10 to $22	$1 to $2	$5 to $8
Project Costs ($/SF) (1)	$4 to $8	$14 to $23	$10 to $30	$20 to $42	$4 to $10	$8 to $16
Overlay Minimum Thickness (Inches)	0.375	1.50	0.75	2.00	2.00	2.00
Wearing Surface Distress (delamination, spalls, or patches)	≤ 2%	≤ 25%	≤ 5%	≤ 25%	NA	≤ 25%
Deck Patch Material	Concrete (3), rapid set (2), or overlay mix	Overlay mix	Concrete (3), rapid set, or PPC	Concrete (3) or rapid set (2)	Concrete (3) or rapid set (2)	Concrete (3) or rapid set (2)
Typical Surface Preparation	Shot blast	Milled and shot blast (5)	Shot blast (5)	Sand blast	Water or air blast	Sand blast (5)
Overlay Finish	Aggregates	Tined	Tined and sanded	None	None	None

(1) Estimated costs based on CY2017 and is for informational pursues only. Overlay costs includes minimum overlay thickness and overlay placement costs. Project costs includes all structure associated costs (joint repairs, deck repairs, surface preparations, minimum overlay thickness). Costs do not include traffic control costs or other costs not captured on structure costs.
(2) Requires approval
(3) Portland cement concrete patch material may require a 28-day cure prior to overlay placement.
(4) Not eligible for federal funds
(5) 1 to 3/4-inch milling recommended for decks exposed longer than 10 years and not previously milled
(6) Estimated durations based on the overlay placement time to the minimum time until traffic can to be placed on the overlay. Durations do not include time for deck repairs or staging considerations.

Table 40.5-1
Overlay Selection Considerations

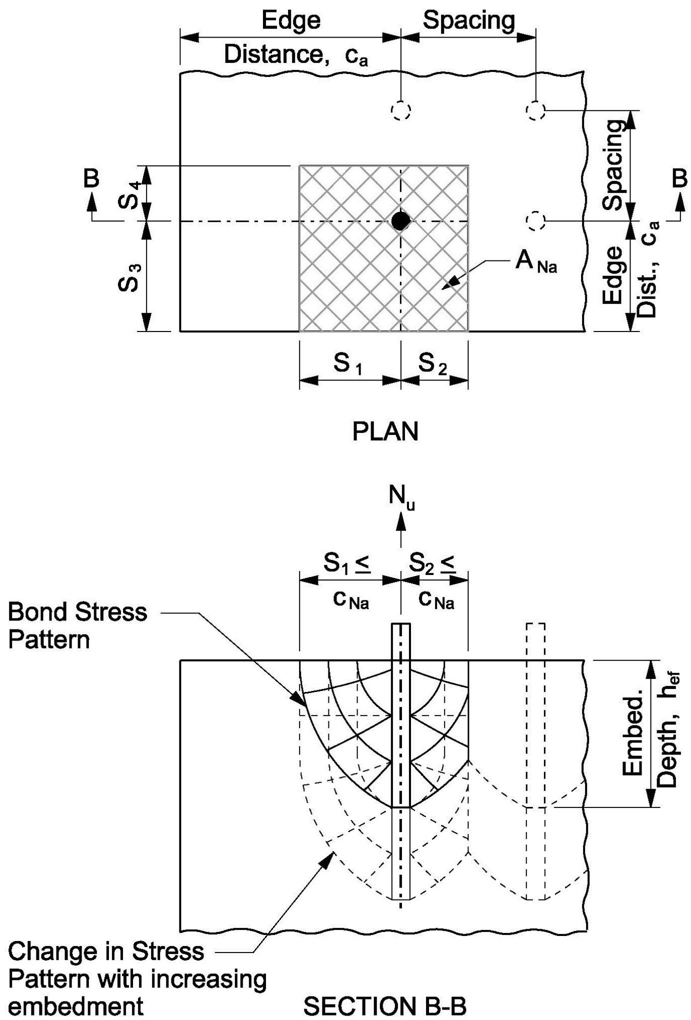

Figure 40.16-2
Bond Failure of Concrete Adhesive Anchors in Tension

The projected influence area of a single adhesive anchor, A_{Na}, is shown in Figure 40.16-2. Unlike the concrete breakout area, it is not affected by the embedment depth of the anchor. A_{Na} is limited in each direction by S_i:

S_i = Minimum of:

1. $c_{Na} = 10 d_a \sqrt{\frac{\tau_{uncr}}{1100}}$,

2. Half of the spacing to the next anchor in tension, or

3. The edge distance (c_a) (in).

Anchor Size, d_a	Adhesive Anchors			
	Dry Concrete		Water-Saturated Concrete	
	Min. Bond Stress, τ_{uncr} (psi)	Min. Bond Stress, τ_{cr} (psi)	Min. Bond Stress, τ_{uncr} (psi)	Min. Bond Stress, τ_{cr} (psi)
#4 or 1/2"	990	670	370	280
#5 or 5/8"	970	720	510	410
#6 or 3/4"	950	580	500	420
#7 or 7/8"	930	580	490	420
#8 or 1"	770	580	600	490

Table 40.16-1
Tension Design Table for Concrete Anchors

The minimum bond stress values for adhesive anchors in Table 40.16-1 are based on the Approved Products List for "Concrete Adhesive Anchors". The designer shall determine whether the concrete adhesive anchors are to be utilized in dry concrete (i.e., rehabilitation locations where concrete is fully cured, etc.) or water-saturated concrete (i.e., new bridge decks, box culverts, etc.) and shall design the anchors accordingly.

The factored tension force on each anchor, N_u, must be less than or equal to the factored tensile resistance, N_r. For mechanical anchors:

$$N_r = \phi_{ts} N_{sa} \le \phi_{tc} N_{cb} \le \phi_{tc} N_{pn}$$

In which:

ϕ_{ts} = Strength reduction factor for anchors in concrete, **ACI [17.3.3]**
= 0.65 for brittle steel as defined in 40.16.1.1
= 0.75 for ductile steel as defined in 40.16.1.1

N_{sa} = Nominal steel strength of anchor in tension, **ACI [17.4.1.2]**
= $A_{se,N} f_{uta}$

$A_{se,N}$ = Effective cross-sectional area of anchor in tension (in^2)

f_{uta} = Specified tensile strength of anchor steel (psi)

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41.6 Programming Policy for Structures Improvement Projects

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

41.6.1 Bridge Age

WisDOT policy item:

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

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

41.6.2 Bridge Ratings

WisDOT policy item:

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

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

41.6.3 Vertical Clearance

WisDOT policy item:

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

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

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

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

41.6.4 Hydraulics

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

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

41.6.5 Freight Considerations

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

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

41.6.6 Cost Benefit Analysis

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

41.6.6.1 Treatment Schedule

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

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the most recent inspection reports and consider the current state of deterioration when load rating a bridge.

45.2.4 Coupling Design with In-Service Loading

As discussed above, design live load vehicles have evolved through the years in an attempt to accurately represent actual in-service traffic. However, until the mid-1950s, there was no legislative connection between the size and weight of in-service traffic and the design capacity of the nation’s bridges. Put more simply, with some local or regional exceptions, it was generally legal to drive any size truck, anywhere. In 1956, this began to change. Congress legislated limits on maximum axle weight (18,000 lbs. on a single axle, 32,000 lbs. for a tandem axle), and gross weight (73,280 lbs.), though there were “grandfather” provisions included. However, even with these limitations, it was still very possible to have a vehicle configuration deemed legal according to the above provisions, but that would induce force effects in excess of the bridge design capacity. Arguably the most significant change in truck size and weight legislation came in 1974 when Congress established the Federal Bridge Formula. The Federal Bridge Formula remains the foundation of truck size and weight legislation today.

45.2.5 Federal Bridge Formula

In the late 1950s, AASHTO conducted an extensive series of field tests to study the effects of truck traffic on pavements and bridges. Based on these tests and an extensive structural analysis effort, the Federal Bridge Formula was developed. The formula is intended to limit the weights of shorter trucks to levels which will limit the overstress in well-maintained bridges designed with HS-20 loading to about 3% and in well-maintained HS-15 bridges to about 30%. While often displayed in table format, the actual formula is as follows.

$$W = 500\{\left[\frac{LN}{N-1}\right] + 12N + 36\}$$

Where: W = the maximum weight in pounds that can be carried on a group of two or more axles to the nearest 500 lbs.

L = the spacing in feet between the outer axles of any two or more axles

N = the number of axles being considered

There are numerous resources readily available to more extensively explain the use of the formula, but it’s important to note that the allowable weight is dependent on the number of axles and the axle spacing. In general, the Federal Bridge Formula is the basis of defining a legal-weight vehicle configuration in Wisconsin. Unless specifically covered via state statute, vehicles that do not conform to the formula must apply for a permit in order to travel over bridges in the Wisconsin. Over-weight truck permitting is discussed further in 45.11. When it is determined that a bridge is not able to safely carry the legal-weight loads, the structure must be load posted. Load postings are discussed in more detail in 45.10.

45.3 Load Rating Process

The following section provides direction on general policies and procedures related to the process for developing a bridge load rating for WisDOT.

45.3.1 Load Rating a New Bridge (New Bridge Construction)

New bridges shall be rated using Load and Resistance Factor Rating (LRFR) methodology. See 45.3.6 for a discussion on rating methodologies.

45.3.1.1 When a Load Rating is Required (New Bridge Construction)

It is mandatory for all new bridges to be load rated. Bridges being analyzed for staged construction shall satisfy the requirements of LRFR for each construction stage. For staged construction, utilize the same load factors, resistance factors, load combinations, etc. as required for the final configuration, unless approved by the WisDOT Bureau of Structures Rating Unit.

45.3.2 Load Rating an Existing (In-Service) Bridge

If an existing bridge was designed using LRFD methodology, it shall be rated using LRFR.

If an existing bridge was designed using Load Factor Design (LFD) methodology, it shall be rated using Load Factor Rating (LFR). It is also acceptable to rate using LRFR, but this shall be approved in advance by the WisDOT Bureau of Structures Rating Unit.

If an existing bridge was designed using Allowable Stress Design (ASD) methodology, it shall be rated using LFR. It is also acceptable to rate using LRFR, but this shall be approved in advance by the WisDOT Bureau of Structures Rating Unit. There is an exception for bridges with timber or concrete masonry superstructures. For these types only, it is acceptable to utilize Allowable Stress Rating (ASR). See 45.3.6 for a discussion on rating methodologies.

Bridges being analyzed for staged construction during a rehabilitation project shall satisfy the requirements of the appropriate rating methodology (LRFR, LFR, or ASR) for each construction stage. Utilize the same load factors, resistance factors, load combinations, etc. as required for the final configuration, unless approved by the WisDOT Bureau of Structures Rating Unit.

Consultants are required to investigate the level of effort required for a given load rating prior to negotiating a contract with WisDOT. This is critical in order to accurately estimate the number of hours required for the load rating. It is also strongly recommended that the rating analysis be performed as early as possible for a rehabilitation project, in the case the ratings are unexpectedly low or weight limit restrictions are required (including annual permits or emergency vehicles), and the scope of the project requires adjustment in order to improve the ratings.

45.3.2.1 When a Load Rating is Required (Existing In-Service Bridge)

WisDOT policy items:

The load rating effort for rehabilitation projects is intended to be independent of previous ratings. Previous analysis files should be used for information and verification purposes only.

Bridges shall be load rated for any project that results in a change in the loads applied to a structure or to an individual structural element that would typically require a load rating (See 45.3.3 for requirements on what elements should be rated). This requirement includes any of (but is not limited to) the following activities:

- Superstructure replacement
- Deck replacement
- Deck overlays
    - New overlays – concrete, asphalt, or polymer
    - Removal of existing overlays and placement of a new overlay
- Bridge widenings
- Superstructure alterations (re-aligning girders, adding girders, etc.)

(Note: WisDOT recognizes that some of the activities noted above may not result in an appreciable change to the load rating. However, it is WisDOT policy to use these instances as an opportunity for quality control of the load rating for that structure and to verify that the load rating takes into account any current deterioration.)

Bridges shall be load rated if there is noted (inspection reports or otherwise) a significant change in the ability of a member to carry load, i.e. deterioration or distortion.

Bridges require a load rating assessment due to impact damage. This assessment may not necessarily include a re-calculation of the load rating if the damage is deemed to be minimal by a qualified engineer.

45.3.3 What Should be Rated

In general, primary load-carrying members are required to be load rated. Secondary elements may be load rated if there is significant deterioration or if there is question regarding the original design capacity. The load rating engineer is responsible for the decision on load rating secondary elements.

If the load rating engineer, utilizing engineering judgment, determines that certain members or components will not control the rating, then a full analysis of the non-controlling element is not required. Justification for member selection should be clearly stated in the load rating calculations submitted to WisDOT Bureau of Structures. See 45.9 for more information on submittal requirements.

45.3.3.1 Superstructure

- Steel Girder Structures

Primary elements for rating include girders (interior and exterior), floorbeams (if present), and stringers (if present). The concrete deck as it relates to any composite action with the girder (and potentially reinforcing steel in the deck for negative moment applications), is also part of the primary system. While cross frames are considered primary members in a curved girder structure or steel tub girder, these members are not considered to be controlling members, and do not need to be analyzed for load rating purposes. If the inspection report indicates signs of distortion or buckling, the cross frame shall be evaluated and the effects on the adjacent girders considered.

Shiplap joints (if present), and pin-and-hanger joints (if present) also may be considered primary elements. Contact the Bureau of Structures Rating Unit to discuss load ratings for these elements.

Secondary elements include bolted web or flange splices, cross frames and/or diaphragms, stringer-to-floorbeam connections (if present), and floorbeam-to-girder connections (if present).

- Prestressed Concrete Girder Structures

Primary elements for rating include prestressed girders (interior and exterior). The concrete deck (and potentially reinforcing steel in the deck for negative moment applications), as it relates to any composite action with the girder, is also part of the primary system.

Secondary elements include diaphragms.

- Concrete Slab Structures

Primary elements for rating include the structural concrete slab. For design of new concrete slabs or rehabilitation of existing concrete slabs, load ratings reported on plans shall include both interior and exterior slab strips. However, for rating in-service concrete slab structures, exterior slab strip ratings are not required if the exterior strip does not show signs of distress and heavy truck loads are expected to travel within the striped lanes (see 45.5.1.2).

Another primary element for rating could include an integral concrete pier cap, if there is no pier cap present. This would take the form of increased transverse reinforcement at the pier, likely combined with a haunched slab design.

- Steel Truss Structures

Primary elements for rating include truss chord members, truss diagonal members, gusset plates connecting truss chord or truss diagonal members, floor beams (if present), and stringers (if present). If any panel points of the truss were designed as braced, bracing members and connections may be considered primary elements.

Secondary elements include splices, stringer-to-floorbeam connections (if present), floorbeam-to-truss connections (if present), lateral bracing, and any gusset plates used to connect secondary elements.

- Timber Girder or Slab Structures

Primary elements for rating include timber girders or timber slab members.

Secondary elements include diaphragms (solid sawn or cross-bracing), stiffener beams, or any tie rods that are present.

- Concrete Box or Channel Structures

Primary elements for rating include concrete box girders.

Secondary elements include diaphragms and shiplap joint connections (if present).

- Additional Elements and Other Structures Types

Transfer girders, straddle bents and/or integral pier caps are considered primary elements. If these elements are present supporting the superstructure to be rated, they are to be included in the load rating.

Other superstructure types should be load rated based on the judgment of the load rating Engineer of Record. The structure types noted below most likely require refined analysis methods to accurately determine the controlling load rating. See 45.3.11 for WisDOT guidance on refined analysis.

- Steel arch
- Curved or kinked steel girder
- Steel tub girder
- Rigid frame structure (steel or concrete)
- Steel bascule or vertical lift
- Cable-stayed or suspension
- Other more complex structure types that may require efforts beyond typical line girder analysis

As with more typical superstructure types, the load rating engineer should thoroughly review inspection reports when making the decision on what superstructure elements may require a load rating.

45.3.3.2 Substructure

Substructures generally do not control the load rating. Scenarios where substructure element conditions may prompt a load rating include, but are not limited to:

- Collision or impact damage
- Substructure components with significant deterioration, particularly those with a lack of redundancy
- Scour, undermining, or settlement which may affect a footing's bearing capacity or a column's unbraced length

WisDOT policy items:

Reinforced concrete piers are not typically rated. However, if the pier – and particularly the pier cap - has large cracks, significant spalling, or exposed reinforcement that shows deterioration, a more thorough evaluation may be appropriate. Reinforced concrete pier caps exhibiting signs of shear cracks may also warrant further evaluation.

In general, reinforced concrete abutments do not require a load rating. However, if the abutment has large cracks, tipping, displacement, or other movement, a more thorough evaluation may be appropriate.

In either of the cases above, contact the Bureau of Structures Rating Unit to discuss the level of effort required for evaluation.

- Extensive section loss from corrosion or rot. WisDOT recommends reviewing inspection reports and paying particular attention for the following scenarios:
    - Exposed steel pile bents
    - Exposed steel pile abutments
    - Exposed timber pile bents
    - Exposed timber pile abutments
    - Exposed timber pile caps

Based on experience, WisDOT has found the above elements to be particularly susceptible to deterioration, particularly if wet conditions are present. If deterioration is significant, these substructure members may control the rating. In the case of timber piles, calculated ratings may be low, even with little or no deterioration. See 45.7.1 for further discussion on timber piles.

The load rating engineer should thoroughly review inspection reports when making the decision on what substructure elements may require a load rating.

45.3.3.3 Deck

Reinforced concrete decks on redundant, multi-girder bridges are not typically load rated. A load rating would only be required in cases of significant deterioration, damage, or to investigate particularly heavy wheel or axle loads. A deck designed using an antiquated design load (H-10, H-15, etc.) may also warrant a load rating.

Other deck types (timber, filled corrugated steel) generally have lower capacity than reinforced concrete decks. This should be taken under consideration when load rating a structure with one of these deck types. Other deck types may also be more susceptible to damage or deterioration.

It is the responsibility of the load rating engineer to determine if a load rating for the deck is required.

45.3.4 Data Collection

Proper and complete data collection is essential for the accurate load rating of a bridge. It is the responsibility of the load rating engineer to gather all essential data and to assess its reliability. When assumptions are used, they should be noted and justified.

45.3.4.1 Existing Plans

Existing design plans are used to determine original design loads, bridge geometry, member section properties, and member material properties. It is important to review all existing plans; original plans as well as plans for any rehabilitation projects (deck replacements, overlays, etc.). If possible, as-built plans should be consulted as well. These plans reflect any changes made to the design plans during construction. Repair plans that document past repairs to the structure may also be available and should be reviewed, if they exist.

If no plans exist or if existing plans are illegible, field measurements may be required to determine bridge geometries and member section properties. Assumptions may have to be made on material properties. Direction on material assumptions is addressed in 45.5.2.

45.3.4.2 Shop Drawings and Fabrication Plans

Shop drawings and fabrication plans can be an extremely valuable source of information when performing a load rating. Shop drawings and fabrication plans are probably the most accurate documentation of what members and materials were actually used during construction, and may contain information not found in the design plans.

WisDOT has an inventory of shop drawings and fabrication plans, but they do not exist for every existing bridge. If the load rating engineer feels shop drawings and/or fabrication plans are required in order to accurately perform the load rating, contact the Bureau of Structures Rating Unit for assistance.

45.3.4.3 Inspection Reports

When rating an existing bridge, it is critical to review inspection reports, particularly the most recent report. Any notes regarding deterioration, particularly deterioration in primary load-carrying members, should be paid particular attention. It is the responsibility of the load rating engineer to evaluate any recorded deterioration and determine how to properly model that deterioration in a load rating analysis. Reviewing historical inspection reports can offer insight as to the rate of growth of any reported deterioration. Inspection reports can also be used to verify existing overburden.

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 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 based on 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 (HSIS), an on-line bridge management system developed by internally by WisDOT. For more information on HSIS, see 45.3.5. For questions on inspection-related issues, please contact the Bureau of Structures Maintenance Section.

45.3.4.4 Other Records

Other records may exist that can offer additional information or insight into bridge design, construction, or rehabilitation. In some cases, these records may override information found in design plans. It is the responsibility of the load rating engineer to gather all pertinent information and decide how to use that information. Examples of records that may exist include:

- Standard plans – generic design plans that were sometimes used for concrete t-girder structures, concrete slab structures, steel truss structures, and steel through-girder structures.
- Correspondences
- Material test reports
- Mill reports
- Non-destructive test reports
- Photographs
- Repair records
- Historic rating analysis

Once a bridge has been removed, records are removed from HSIS. However, if the bridge was removed after 2003, information may still be available by contacting the Bureau of Structures Bridge Management Unit.

45.10.3.2 Legal Load Rating Load Posting Equation (LRFR)

When using the LRFR method and the operating rating factor (RF) calculated for each legal truck described above is greater than 1.0, the bridge does not need to be posted. When for any legal truck the RF is between 0.3 and 1.0, then the following equation should be used to establish the safe posting load for that vehicle (see **MBE [Equation 6A8.3-1]**):

$$\text{Posting} = \frac{W}{0.7}\left[(RF) - 0.3\right]$$

Where:

RF	=	Legal load rating factor
W	=	Weight of the rating vehicle

When the rating factor for any vehicle type falls below 0.3, then that vehicle type should not be allowed on the bridge. If necessary, the structure may need to be closed until it can be repaired, strengthened, or replaced. This formula is only valid for LRFR load posting calculations.

45.10.3.3 Distribution Factors for Load Posting Analysis

WisDOT policy items:

The AASHTO Commercial Vehicles, Specialized Hauling Vehicles, and Emergency Vehicles shall be analyzed using a multi-lane distribution factor for bridge widths 18'-0" or larger. Single lane distribution factors are used for bridge widths less than 18'-0".

The WisDOT Specialized Annual Permit Vehicles shown in Figure 45.10-3 shall be analyzed using a single-lane distribution factor, regardless of bridge width.

The Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed for load postings using a multi-lane distribution factor for bridge widths 18'-0" or larger. Single lane distribution factors are used for bridge widths less than 18'-0".

For Specialized Hauling Vehicles, single-lane distribution factor may be considered on two-lane roadways with travel in opposite directions to avoid a new or reduced load posting, if the bridge has demonstrated an ability to carry routine legal loads in its vicinity. Contact the Bureau of Structures Rating Unit for approval to use single-lane distribution factors on bridges with multiple lanes.

For Emergency Vehicles, refined analysis may be used to determine alternative distribution factors based on only one EV in one lane loaded simultaneously with other unrestricted legal vehicles in other lanes. This exception will reduce the computed load effects and yield higher load ratings. Refer to FHWA's "Questions and Answers: Load Rating for the FAST Act's Emergency Vehicles, Revision R01" (March 2018).

45.10.4 Load Posting Signage

Current WisDOT policy is to post State bridges for a single gross weight, in tons. Bridges that cannot carry the maximum weight for the vehicles described in 45.10.2 at the operating level are posted with the standard sign shown in Figure 45.10-6. This sign shows the bridge capacity for the governing load posting vehicle, in tons. The sign should conform to the requirements of the *Wisconsin Manual for Uniform Traffic Control Devices (WMUTCD)*.

In the past, local bridges were occasionally posted with the signs shown in Figure 45.10-7 using the H20, Type 3 and Type 3S2 vehicles. The H20 represented the two-axle vehicle, the Type 3 represented the three-axle vehicle and the Type 3S2 represented the combination vehicle. This practice is not encouraged by WisDOT and is generally not allowed for State-owned structures, except with permission from the State Bridge Maintenance Engineer.

Emergency vehicle posting signs, however, are based on a combination of the single axle, tandem axle, and gross vehicle weight limits, as shown in Figure 45.10-8. Emergency vehicle posting signs are only required for bridges on the Interstate and within reasonable access (one road mile) to or from an Interstate interchange.

WEIGHT LIMIT 10 TONS	BRIDGE CLOSED

Figure 45.10-6

Standard Signs Used for Posting Bridges

WEIGHT LIMIT 2 AXLE VEHICLES 15 TONS 3 AXLE VEHICLES 20 TONS COMBINATION VEHICLES 30 TONS	WEIGHT LIMIT 2 AXLE VEHICLES 14 TONS 3 AXLE VEHICLES 18 TONS COMBINATION VEHICLES 28 TONS	WEIGHT LIMIT 2 AXLE VEHICLES 14 TONS 3 AXLE VEHICLES 18 TONS COMBINATION VEHICLES 28 TONS

Figure 45.10-7

Historic Load Posting Signs

EMERGENCY	
VEHICLE	
WEIGHT LIMIT	
SINGLE AXLE	15 TONS
TANDEM	25 TONS
GROSS	35 TONS

Figure 45.10-8
Emergency Vehicle Load Posting Signs

45.11 Over-Weight Truck Permitting

45.11.1 Overview

Size and weight provisions for vehicles using the Wisconsin network of roads and bridges are specified in the Wisconsin Statutes, Chapter 348: Vehicles – Size, Weight and Load. Weight limits for legal-weight traffic and over-weight permit requirements are defined in detail in this chapter. The webpage for Chapter 348 is shown below.

https://docs.legis.wisconsin.gov/statutes/statutes/348

Over-weight permit requests are processed by the WisDOT Oversize Overweight (OSOW) Permit Unit in the Bureau of Highway Maintenance. The permit unit collaborates with the WisDOT Bureau of Structures Rating Unit to ensure that permit vehicles are safely routed on the Wisconsin inventory of bridges.

While the Wisconsin Statutes contain several industry-specific size and weight annual permits, in general, there are two permit types in Wisconsin: multi-trip (annual) permits and single-trip permits.

45.11.2 Multi-Trip (Annual) Permits

Multi-trip permits are granted for non-divisible loads such as machines, self-propelled vehicles, mobile homes, etc. They typically allow unlimited trips and are available for a range of three months to one year. The permit vehicle may mix with typical traffic and move at normal speeds. Multi-trip permits are required to adhere to road and bridge load postings and are subject to additional restrictions based on restricted bridge lists supplied by the WisDOT Bureau of Structures Rating Unit and published by the WisDOT OSOW Permit Unit. The restricted bridge lists are developed based on the analysis of the Wisconsin Standard Permit Vehicle (Wis-SPV). For more information on the Wis-SPV and required analysis, see 45.12. The carrier is responsible for their own routing, and are required to avoid these restrictions and load postings.

Vehicles applying for a multi-trip permit are limited to 170,000 pounds gross vehicle weight, plus additional restrictions on maximum length, width, height, and axle weights. Please refer to the WisDOT Oversize Overweight (OSOW) Permits website or the Wisconsin Statues (link above) for more information.

https://www.dot.wisconsinwisconsindot.gov/business/carriers/osowgeneral.htm

45.11.3 Single Trip Permits

Non-divisible loads which exceed the annual permit restrictions may be moved by the issuance of a single trip permit. When a single trip permit is issued, the applicant is required to indicate on the permit the origin and destination of the trip and the specific route that is to be used. A separate permit is required for access to local roads. Each single trip permit vehicle is individually analyzed by WisDOT for all state-owned structures that it encounters on the designated permit route.

Live load distribution for single trip permit vehicles is based on single lane distribution. This is used because these permit loads are infrequent and are likely the only heavy loads on the structure during the crossing. The analysis is performed at the operating level.

At the discretion of the engineer evaluating the single trip permit, the dynamic load allowance (or impact for LFR) may be neglected provided that the maximum vehicle speed can be reduced to 5 MPH prior to crossing the bridge and for the duration of the crossing.

In some cases, the truck may be escorted across the bridge with no other vehicles allowed on the bridge during the crossing. If this is the case, then the live load factor (LRFR analysis) can be reduced from 1.20 to 1.10 as shown in Table 45.3-3. It is recommended that the truck be centered on the bridge if it is being escorted with no other vehicles allowed on the bridge during the crossing.

Vehicles with non-standard axle gauges may also receive special consideration. This may be achieved by performing a more-rigorous analysis of a given bridge that takes into account the specific load configuration of the permit vehicle in question instead of using standard distribution factors that are based on standard-gauge axles. Alternatively, modifications may be made to the standard distribution factor in order to more accurately reflect how the load of the permit vehicle is transferred to the bridge superstructure. How non-standard gauge axles are evaluated is at the discretion of the engineer evaluating the permit.

45.12 Wisconsin Standard Permit Vehicle (Wis-SPV)

45.12.1 Background

The Wis-SPV configuration is shown in Figure 45.12-1. It is an 8-axle, 190,000lbs vehicle. It was developed through a Wisconsin research project that investigated the history of multi-trip permit configurations operating in Wisconsin. The Wis-SPV was designed to completely envelope the force effects of all multi-trip permit vehicles operating in Wisconsin and is used internally to help regulate multi-trip permits.

45.12.2 Analysis

- New Bridge Construction

For any new bridge design, the Wis-SPV shall be analyzed. The Wis-SPV shall be evaluated at the operating level. When performing this design check for the Wis-SPV, the vehicle shall be evaluated for single-lane distribution assuming that the vehicle is mixing with normal traffic and that the full dynamic load allowance is utilized. For this design rating, a future wearing surface shall be considered. Load distribution for this check is based on the interior strip or interior girder and the distribution factors given in Section 17.2.7, 17.2.8, or 18.4.5.1 where applicable. See also the WisDOT policy item in 45.3.7.8.1.

For LRFR, the Wis-SPV design check shall be a permit load rating and shall be evaluated for the limit states noted in Table 45.3-1 and Table 45.3-3.

The design engineer shall check to ensure the design has a RF > 1.0 (gross vehicle load of 190 kips) for the Wis-SPV. If the design is unable to meet this minimum capacity, the engineer is required to adjust the design until the bridge can safely handle a minimum gross vehicle load of 190 kips.

Results of the Wis-SPV analysis shall be reported per 45.9.

- Bridge Rehabilitation Projects

For rehabilitation design, analysis of the Wis-SPV shall be performed as described above for new bridge construction. All efforts should be made to obtain a RF > 1.0 (gross vehicle load of 190 kips) within the confines of the scope of the project. However, it is recognized that it may not be possible to increase the Wis-SPV rating without a significant change in scope of the project. <u>In these cases, consult the Bureau of Structures Rating Unit for further direction.</u>

Results of the Wis-SPV analysis shall be reported per 45.9.

- Existing (In-Service) Bridges

When performing a rating for an existing (in-service) bridge, analysis of the Wis-SPV shall be performed as described above for new bridge construction. In this case – where the bridge in question is being load rated but not altered in any way – the results of the Wis-SPV analysis need simply be reported as calculated per 45.9. <u>If the results of this analysis produce a rating</u>

factor less than 1.0 (gross vehicle load less than 190 kips), notify the Bureau of Structures Rating Unit.

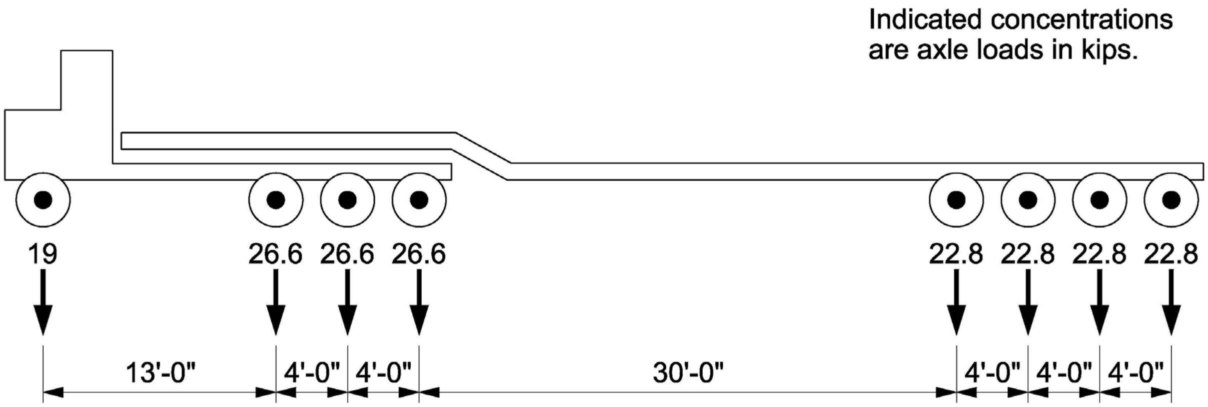

Figure 45.12-1
Wisconsin Standard Permit Vehicle (Wis-SPV)

45.13 References

1. *Final Report on Full-Scale Bridge Testing* by E. G. Burdette and D. W. Goodpasture, Department of Civil Engineer, University of Tennessee, 1971.

2. *The AASHTO Road Test*, Report 4 Bridge Research by the Highway Research Board, Washington, D.C. 1962.

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

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

5. *The Manual for Bridge Evaluation, 2015 Interim Revisions* by American Association of State Highway and Transportation Officials, 2015.

6. *Structure Inspection Manual* by Wisconsin Department of Transportation, 2003.

7. *Reinforced Concrete Design* by C. K. Wang and C. G. Salmon.

8. *Plastic Design of Steel Frames* by Lynn S. Beedle.

9. National Cooperative Highway Research Program Report 312.

10. National Cooperative Highway Research Program Project 12-63.

11. *Post-Tensioning Manual* by Post-Tensioning Institute.

12. Wisconsin Statutes, Vol. 4, Chapter 348.

13. *Summary of Motor Vehicle Size and Weight Regulations in Wisconsin* by Dept. of Transportation, Division of Motor Vehicles.

14. *Evolution of Vehicular Live Load Models During the Interstate Design Era and Beyond*, John M. Kulicki and Dennis R. Mertz, Transportation Research Circular; 50 Years of Interstate Structures.

15. *Bridge Inspector's Reference Manual;* Federal Highway Administration.

16. *The Collapse of the Silver Bridge*, Chris LeRose; www.wvculture.org/history/wvhs1504.html

17. Engineering News, September 1914; L.R. Manville and R.W. Gastmeyer

18. *AASHTO Guide Specifications for Horizontally Curved Steel Girder Highway Bridges*, by American Association of State Highway and Transportation Officials, 2003.

19. *G13.1 Guidelines for Steel Girder Bridge Analysis* by American Association of State Highway and Transportation Officials and by National Steel Bridge Alliance (NSBA), 2nd Ed., 2014.

20. *Questions and Answers: Load Rating for the FAST Act's Emergency Vehicles, Revisions R01* by Federal Highway Administration Office of Bridges and Structures, 2018.

21. *Load Rating for the FAST Act Emergency Vehicles EV-2 and EV-3*, National Cooperative Highway Research Program Project 20-07 / Task 410, 2019.

45.14 Rating Examples

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E45-5 Reinforced Concrete Slab Rating Example - LFR

Reference E45-1 for bridge data. For LFR, the Bureau of Structures rates concrete slab structures for the Design Load (HS20) and for Permit Vehicle Loads on an interior strip equal to one foot width.

This example calculates ratings of the controlling locations at the 0.4 tenths point of span 1 for positive moment and at the pier for negative moment.

E45-5.1 Design Criteria

Geometry:

$L_1 := 38.0\ \text{ft}$	Span 1 Length
$L_2 := 51.0\ \text{ft}$	Span 2 Length
$L_3 := 38.0\ \text{ft}$	Span 3 Length
$slab_{width} := 42.5\ \text{ft}$	out to out width of slab
$cover_{top} := 2.5\ \text{in}$	concrete cover on top bars (includes 1/2 in wearing surface)
$cover_{bot} := 1.5\ \text{in}$	concrete cover on bottom bars
$d_{slab} := 17\ \text{in}$	slab depth (not including 1/2 in wearing surface)
$b := 12\ \text{in}$	interior strip width for analysis
$D_{haunch} := 28\ \text{in}$	haunch depth (not including 1/2 in wearing surface)
$A_{st\_0.4L} := 1.71\ \text{in}^2$	area of longitudinal bottom steel at 0.4L (#9's at 7 in centers) per foot slab width
$A_{st\_pier} := 1.88\ \text{in}^2$	area of longitudinal top steel at Pier (#8's at 5 in centers) per foot slab width

Material Properties:

$f'_c := 4\ \text{ksi}$	concrete compressive strength
$f_y := 60\ \text{ksi}$	yield strength of reinforcement

Weights:

$w_c := 150\ \text{pcf}$ concrete unit weight

$w_{LF} := 387\ \text{plf}$ weight of Type LF parapet (each)

E45-5.2 Analysis of an Interior Strip - one foot width

Use Strength Limit States to rate the concrete slab bridge. **MBE [6B.5.3.2]**

E45-5.2.1 Dead Loads

The slab dead load, D_{slab}, and the section properties of the slab, do not include the 1/2 inch wearing surface. But the 1/2 inch wearing surface load, D_{WS}, of 6 psf must be included in the analysis of the slab. For a one foot slab width:

$D_{WS} := 6\ \text{plf}$ 1/2 inch wearing surface load

The parapet dead load is uniformly distributed over the full width of the slab when analyzing an Interior Strip. For a one foot slab width:

$$D_{para} := 2 \cdot \frac{w_{LF}}{slab_{width}} \cdot 1\ \text{ft} = 18\ \text{plf}$$

The unfactored dead load moments, M_D, due to slab dead load (D_{slab}), parapet dead load (D_{para}), and the 1/2 inch wearing surface (D_{WS}) are shown in Chapter 18 Example E18-1 (Table E18.4). For LFR, the total dead load moment (M_D) is the sum of the values M_{DC} and M_{DW} tabulated separately for LRFD calculations.

The structure was designed for a possible future wearing surface, D_{FWS}, of 20 psf.

$D_{FWS} := 20\ \text{plf}$ possible future wearing surface per foot slab width

E45-5.2.2 Live Load Distribution

Live loads are distributed over an equivalent width, E, as calculated below.

The live loads are to be placed on these widths are wheel loads (i.e., one line of wheels) or half of the lane load. The equivalent distribution width applies for both live load moment and shear.

Multi-Lane Loading:	E = 48.0 in + 0.06 S	$\le$ 84 in	**Std [3.24.3.2]**
Single-Lane Loading:	E = (12/7) • (48.0 in + 0.06 S)	$\le$ 144 in	**[45.6.2.1]**

where:

S = effective span length, in inches

For multi-lane loading:

(Span 1, 3) $E_{m13} := min\left(84\ \text{in}, 48\ \text{in} + 0.06 \cdot L_1\right) = 75.4\ \text{in}$

(Span 2) $E_{m2} := min\left(84\ \text{in}, 48\ \text{in} + 0.06 \cdot L_2\right) = 84\ \text{in}$

For single-lane loading:

(Span 1, 3) $E_{s13} := \frac{12}{7} \cdot E_{m13} = 129.2\ \text{in}$

(Span 2) $E_{s2} := \frac{12}{7} \cdot E_{m2} = 144\ \text{in}$

E45-5.2.3 Nominal Flexural Resistance (Mn):

The depth of the compressive stress block (a) is:

$$a = \frac{A_s \cdot fy}{0.85 \cdot f'_c \cdot b}$$ **Std (8-17)**

For rectangular sections, the nominal moment resistance, M_n (tension reinforcement only), equals:

$$M_n = A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$$ **Std (8-16)**

where:

d_s = slab depth (excluding 1/2 in. wearing surface) - bar clearance - 1/2 bar diameter

Maximum Reinforcement Check

The area of reinforcement to be used in calculating nominal resistance (M_n) shall not exceed 75 percent of the reinforcement required for the balanced conditions.
MBE [6B.5.3.2]

$$\rho_b := 0.85^2 \cdot \left(\frac{f'_c}{f_y}\right) \cdot \frac{87\ \text{ksi}}{87\ \text{ksi} + f_y} = 0.029 \qquad A_{smax} = \rho_b \cdot b \cdot d_s$$

E45-5.2.4 General Load Rating Equation (for flexure)

$$RF = \frac{C - A_1 \cdot M_D}{A_2 \cdot M_L \cdot (1 + I)}$$

MBE [6B.4.1]

where:

$C = \phi \cdot M_n$

$\phi := 0.9$ **Std [8.16.1.2.2]**

$A_1 := 1.3$ for Dead Loads

A_2 = Live Load factor: 2.17 for Inventory, 1.3 for Operating

M_D = Unfactored Dead Load Moments

M_L = Unfactored Live Load Moments

I = Live Load Impact Factor (maximum 30%)

E45-5.2.5 Design Load (HS20) Rating

Equivalent Strip Width (E) and Distribution Factor (DF)

Use the multi-lane wheel distribution width for (HS20) live load.

The distribution factor, DF, is computed for a slab width equal to one foot.

$$DF = \frac{12\ in}{E}$$

Spans 1 & 3:

$$DF_{13} := \frac{12\ \text{in}}{E_{m13}} = 0.159 \qquad \text{wheels / ft-slab}$$

Span 2:

$$DF_2 := \frac{12\ \text{in}}{E_{m2}} = 0.143 \qquad \text{wheels / ft-slab}$$

Live Load Impact Factor (I)

$$I = \frac{50}{L + 125} \qquad \text{(maximum 0.3)}$$

Std [3.8.2.1]

Spans 1 & 3:

$$I_{13} := min\left(0.3, \frac{50\ \text{ft}}{L_1 + 125\ \text{ft}}\right) = 0.3$$

Span 2:

$$I_2 := min\left(0.3, \frac{50\ \text{ft}}{L_2 + 125\ \text{ft}}\right) = 0.284$$

Live Loads (LL)

The live loads shall be determined from live load analysis software using the higher of the HS20 Truck or Lane loads.

Rating for Flexure

$$RF = \frac{\phi \cdot M_n - 1.3 \cdot M_D}{A_2 \cdot M_L \cdot (1 + I)}$$

The Design Load Rating was checked at 0.1 pts. along the structure and at the slab/haunch intercepts. The governing limit state and location for the HS20 load in positive moment is in span 1 at the 0.4 pt.

Span 1 (0.4 pt.)

Flexural capacity:

$$A_{st\_0.4L} = 1.71\ \text{in}^2$$

$$d_s := d_{slab} - cover_{bot} - \frac{9}{16}\ \text{in} = 14.94\ \text{in}$$

$$a := \frac{A_{st\_0.4L} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 2.51\ \text{in}$$

$$A_{smax} := \rho_b \cdot b \cdot d_s = 5.110 \text{ in}^2$$

$$M_n := A_{st\_0.4L} \cdot f_y \cdot \left(d_s - \frac{a}{2}\right) = 117.0 \text{ kip} \cdot \text{ft} \qquad A_{smax} > A_{st\_0.4L} \qquad \text{OK}$$

The dead load consists of the slab self-weight and parapet weight divided evenly along the slab width:

$M_D := 18.1 \text{ kip} \cdot \text{ft}$ (from Chapter 18 Example, Table E18.4)

The positive live load moment shall be the largest caused by the following (from live load analysis software):

Design Lane: 17.48 kip-ft
Design Truck: 24.01 kip-ft

Therefore:

$$M_L := 24.01 \text{ kip} \cdot \text{ft}$$

Inventory:

| $$RF_i := \frac{\phi \cdot M_n - 1.3 \cdot M_D}{2.17 \cdot M_L \cdot (1 + I_{13})} = 1.207$$ **Inventory Rating = HS24**

Operating:

| $$RF_o := \frac{\phi \cdot M_n - 1.3 \cdot M_D}{1.3 \cdot M_L \cdot (1 + I_{13})} = 2.014$$ **Operating Rating = HS40**

Rating for Shear:

Shear rating for concrete slab bridges may be ignored. Bending moment is assumed to control per **Std [3.24.4]**.

The Rating Factors, RF, for Inventory and Operating Rating are shown on the plans and also on the load rating summary sheet.

E45-5.2.6 Permit Vehicle Load Ratings

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed per **[45.12]**.

The bridge shall be analyzed for this vehicle considering both single-lane and multi-lane distribution, and full dynamic load allowance is utilized. Future wearing surface will not be considered.

For a newly designed bridge, an additional check is required. The designer shall ensure that the results of a single-lane analysis utilizing the future wearing surface are great 190 kips MVW.

E45-5.2.6.1 Wis-SPV Permit Rating with Multi Lane Distribution

The Maximum Permit Vehicle Load was checked at 0.1 pts along the structure and at the slab/haunch intercepts. The governing location is the C/L of the Pier.

The distribution width and impact factors are the same as calculated for the HS20 load.

At C/L of Pier

Flexural capacity:

$$A_{st\_pier} = 1.88 \text{ in}^2$$

$$d_{s\_pier} := D_{haunch} - cover_{top} - \frac{8}{16} \text{ in} = 25 \text{ in}$$

$$a\_pier := \frac{A_{st\_pier} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 2.76 \text{ in}$$

$$A_{smax\_pier} := \rho_b \cdot b \cdot d_{s\_pier} = 8.552 \text{ in}^2 \qquad A_{smax} > A_{st\_pier} \qquad \text{OK}$$

$$M_{n\_pier} := A_{st\_pier} \cdot f_y \cdot \left(d_{s\_pier} - \frac{a\_pier}{2} \right) = 222 \text{ kip} \cdot \text{ft}$$

The dead load consists of the slab self-weight and parapet weight divided evenly along the slab width:

$$M_{D\_pier} := 59.2 \text{ kip} \cdot \text{ft}$$ (from Chapter 18 Example, Table E18.4)

From live load analysis software, the live load moment at the C/L of the Pier due to the Wisconsin Standard Permit Vehicle (Wis-SPV) having a gross vehicle load of 190 kips and utilizing the maximum multi-lane distribution (as Spans 1 and 3) is:

$$M_{LSPVm\_pier} := 66.06 \text{ kip} \cdot \text{ft}$$

Annual Permit:

$$RF_{mpermit} := \frac{\phi \cdot M_{n\_pier} - 1.3 \cdot M_{D\_pier}}{1.3 \cdot M_{LSPVm\_pier} \cdot (1 + I_{13})} = 1.10$$

The maximum Wisconsin Standard Permit Vehicle (Wis-SPV) load is:

$$RF_{mpermit} \cdot 190\ \text{kip} = 209\ \text{kip}$$

E45-5.2.6.2 Wis-SPV Permit Rating with Single Lane Distribution w/o FWS

The live load moment at the C/L of Pier due to the Wis-SPV with single-lane loading may be determined by scaling the live load moment from multi-lane loading:

$$M_{LSPVs\_pier} := M_{LSPVm\_pier} \cdot \frac{E_{m13}}{E_{s13}} = 38.54\ \text{kip} \cdot \text{ft}$$

Single-Trip Permit w/o FWS:

$$RF_{spermit} := \frac{\phi \cdot M_{n\_pier} - 1.3 \cdot M_{D\_pier}}{1.3 \cdot M_{LSPVs\_pier} \cdot (1 + I_{13})} = 1.89$$

The maximum Wisconsin Standard Permit Vehicle (Wis-SPV) load is:

$$RF_{spermit} \cdot 190\ \text{kip} = 358\ \text{kip}$$

The Single-Lane MVW for the Wis-SPV is shown on the plans, up to a maximum of 250 kips. This same procedure used for the (Wis-SPV) can also be used when evaluating the bridge for an actual "Single-Trip Permit" vehicle.

E45-5.2.6.3 Wis-SPV Permit Rating with Single Lane Distribution w/ FWS

From Chapter 18 Example, Table E18.4, the applied moment at the pier from the future wearing surface is:

$$M_{DW\_pier} := 4.9\ \text{kip} \cdot \text{ft}$$

Single-Trip Permit w/ FWS:

$$RF_{spermit\_fws} := \frac{\phi \cdot M_{n\_pier} - 1.3 \cdot (M_{D\_pier} + M_{DW\_pier})}{1.3 \cdot M_{LSPVs\_pier} \cdot (1 + I_{13})} = 1.79$$

The maximum Wisconsin Standard Permit Vehicle (Wis-SPV) load is:

$$RF_{spermit\_fws} \cdot 190 \text{ kip} = 340 \text{ kip} \quad > 190 \text{ kip} \qquad \text{OK}$$

E45-5.3 Summary of Rating

Slab - Interior Strip					
Limit State	Design Load Rating		Permit Load Rating (kips)		
	Inventory	Operating	Multi DF w/o FWS	Single DF w/o FWS	Single DF w/ FWS
Flexure	HS24	HS40	209	358	340