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Remarks and Instructions

Remarks:

Instructions:

Page numbers and corresponding sheet-counts are given in the table below to indicate portions of the *Bridge Design Manual* that are to be removed and inserted to accomplish this revision.

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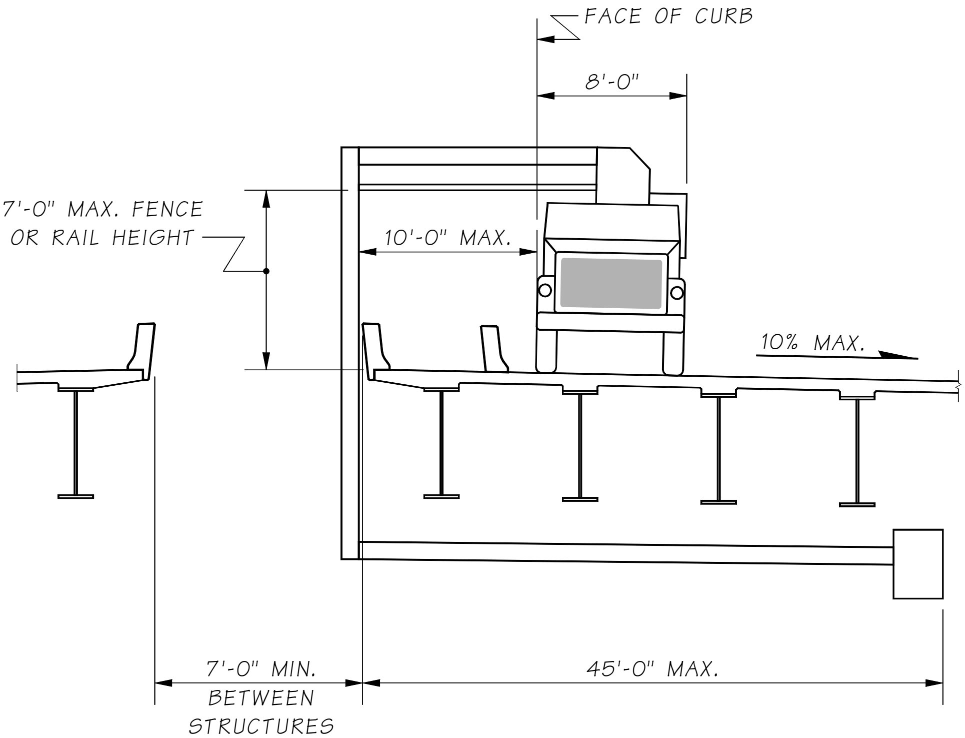


Limits of Under Bridge Inspection Truck
Ubits Can Only Deploy to the Tight Side of Truck
Figure 2.3.11-1

B. Safety Cables

Safety cables strung on steel plate girders or trusses allow for walking access. Care must be given to the application and location. Built-up plate girder bridges are detailed with a safety cable for inspectors walking the bottom flange. However, when the girders become more than 8 feet deep, the inspection of the top flange and top lateral connections becomes difficult to access. It is not feasible for the inspectors to stand on the bottom flanges when the girders are less than 5 feet deep. On large trusses, large gusset plates (3 feet or more wide) are difficult to circumvent. Tie-off cables are best located on the interior side of the exterior girder of the bridge except at large gusset plates. At these locations, cables or lanyard anchors should be placed on the inside face of the truss so inspectors can utilize bottom lateral gusset plates to stand on while traversing around the main truss gusset plates.

C. Travelers

Under bridge travelers, placed on rails that remain permanently on the bridge, can be considered on large steel structures. This is an expensive option, but it should be evaluated for large bridges with high ADT because access to the bridge would be limited by traffic windows that specify when a lane can be closed. Some bridges are restricted to weekend UBIT inspection for this reason.

D. Abutment Slopes

Slopes in front of abutments shall provide enough overhead clearance to the bottom of the superstructure to access bearings for inspection and possible replacement (usually 3 feet minimum).

2.4 Selection of Structure Type

2.4.1 Bridge Types

See Appendix sheet 2.4-A1 for a bar graph comparing structure type, span range and cost range.

The required superstructure depth is determined during the preliminary plan development process. The AASHTO LRFD Specifications in Table 2.5.2.6.3 show traditional minimum depths for constant depth superstructures. WSDOT has developed superstructure depth-to-span ratios based on past experience.

The AASHTO LRFD Specifications, Section 2.5.2.6.1, states that it is optional to check deflection criteria, except in a few specific cases. The WSDOT criteria is to check the live load deflection for all structures as specified in AASHTO LRFD Specifications, section 3.6.1.3.2 and 2.5.2.6.2.

The superstructure depth is used to establish the vertical clearance that is available below the superstructure. For preliminary plans, the designer should use the more conservative depth determined from either the AASHTO LRFD criteria or the WSDOT criteria outlined below. In either case, the minimum depth includes the deck thickness. For both simple and continuous spans, the span length is the horizontal distance between centerlines of bearings.

The superstructure depth may be refined during the final design phase. It is assumed that any refinement will result in a reduced superstructure depth so the vertical clearance is not reduced from that shown in the preliminary plan. However, when profile grade limitations restrict superstructure depth, the preliminary plan designer shall investigate and/or work with the structural designer to determine a superstructure type and depth that will fit the requirements.

A. Reinforced Concrete Slab

1. Application

   Used for simple and continuous spans up to 60 feet.

2. Characteristics

   Design details and falsework relatively simple. Shortest construction time for any cast-in-place structure. Correction for anticipated falsework settlement must be included in the dead load camber curve because of the single concrete placement sequence.

3. Depth/Span Ratios

   a. Constant depth

      Simple span $^1/_{22}$

      Continuous spans $^1/_{25}$

   b. Variable depth

      Adjust ratios to account for change in relative stiffness of positive and negative moment sections.

B. Reinforced Concrete Tee-Beam

1. Application

   Used for continuous spans 30 feet to 60 feet. Has been used for longer spans with inclined leg piers.

2. Characteristics

   Forming and falsework is more complicated than for a concrete slab. Construction time is longer than for a concrete slab.

3. Depth/Span Ratios

   a. Constant depth

      Simple spans $^1/_{13}$

      Continuous spans $^1/_{15}$

   b. Variable depth

      Adjust ratios to account for change in relative stiffness of positive and negative moment sections.

C. Reinforced Concrete Box Girder

1. Application

   Used for continuous spans 50 feet to 120 feet. Maximum simple span 100 feet to limit excessive dead load deflections.

2. Characteristics

   Forming and falsework is somewhat complicated. Construction time is approximately the same as for a tee-beam. High torsional resistance makes it desirable for curved alignments.

3. Depth/Span Ratios*

   a. Constant depth

      Simple spans $^1/_{18}$

      Continuous spans $^1/_{20}$

   b. Variable depth

      Adjust ratios to account for change in relative stiffness of positive and negative moment sections.

      *If the configuration of the exterior web is sloped and curved, a larger depth/span ratio may be necessary.

D. Post-Tensioned Concrete Box Girder

1. Application

   Normally used for continuous spans longer than 120 feet or simple spans longer than 100 feet. Should be considered for shorter spans if a shallower structure depth is needed.

2. Characteristics

   Construction time is somewhat longer due to post-tensioning operations. High torsional resistance makes it desirable for curved alignments.

3. Depth/Span Ratios*

   a. Constant depth

      Simple spans $^1/_{20.5}$

      Continuous spans $^1/_{25}$

b. Variable depth

Two span structures

At Center of span $^1/_{25}$

At Intermediate pier $^1/_{12.5}$

Multi-span structures

At Center of span $^1/_{36}$

At Intermediate pier $^1/_{18}$

*If the configuration of the exterior web is sloped and curved, a larger depth/span ratio may be necessary.

E. Prestressed Concrete Sections

1. Application

Local precast fabricators have several standard forms available for precast concrete sections based on the WSDOT standard girder series. These are versatile enough to cover a wide variety of span lengths.

WSDOT standard girders are:

a. W95G, W83G, WF74G, WF58G, WF50G, WF42G, W74G, W58G, W50G, and W42G precast, prestressed concrete I-girders requiring a cast-in-place concrete roadway deck used for spans less than 180 feet. The number (eq. 95) specifies the girder depth in inches.

W95PTG, W83PTG and WF74PTG post-tensioned, precast segmental I–girders with cast–in–place concrete roadway deck use for simple span up to 164 feet. and continuous span up to 200 feet.

b. U**G* and UF**G* precast, prestressed concrete tub girders requiring a cast–in–place concrete roadway deck are used for spans less than 140 feet. “U” specifies webs without flanges, “UF” specifies webs with flanges, ** specifies the girder depth in inches, and * specifies the bottom flange width in feet. U**G* girders have been precast as shallow as 26 inches.

Post-tensioned, precast, prestressed tub girders with cast–in–place concrete roadway deck are used for simple span up to 164 feet. and continuous span up to 200 feet.

c. W65DG, W53DG, W41DG, and W35DG precast, prestressed concrete decked bulb tee girders requiring an HMA overlay roadway surface used for span less than 150 feet, with the Average Daily Truck (ADT) limitation of 30,000 or less.

d. W62BTG, W50BTG, W38BT6, and W32BTG precast, prestressed concrete bulb tee girders requiring a cast-in-place concrete deck for simple spans up to 120 feet.

e. 12-inch, 18-inch, and 26-inch precast, prestressed slabs requiring 5 inch minimum cast-in-place slab used for spans less than 90 feet.

f. 26-inch precast, prestressed ribbed girder, deck double tee, used for span less than 60 feet, and double tee members requiring an HMA overlay roadway surface used for span less than 40 feet.

2. Characteristics

   Superstructure design is quick for pretensioned girders with proven user-friendly software (PGSuper, PGSplice, and QConBridge)

   Construction details and forming are fairly simple. Construction time is less than for a cast-in-place bridge. Little or no falsework is required. Falsework over traffic is usually not required; construction time over existing traffic is reduced.

   Precast girders usually require that the bridge roadway superelevation transitions begin and end at or near piers; location of piers should consider this. The Region may be requested to adjust these transition points if possible.

   Fully reinforced, composite 8 inch cast-in-place deck slabs continuous over interior piers or reinforced 5 inch cast-in-place deck slabs continuous over interior piers have been used with e. and f.

F. Composite Steel Plate Girder

1. Application

   Used for simple spans up to 260 feet and for continuous spans from 120 to 400 feet. Relatively low dead load when compared to a concrete superstructure makes this bridge type an asset in areas where foundation materials are poor.

2. Characteristics

   Construction details and forming are fairly simple Construction time is comparatively short. Shipping and erecting of large sections must be reviewed. Cost of maintenance is higher than for concrete bridges. Current cost information should be considered because of changing steel market conditions.

3. Depth/Span Ratios

   a. Constant depth

      Simple spans $^1/_{22}$

      Continuous spans $^1/_{25}$

   b. Variable depth

      @ Center of span $^1/_{40}$

      @ Intermediate pier $^1/_{20}$

G. Composite Steel Box Girder

1. Use

   Used for simple spans up to 260 feet and for continuous spans from 120 to 400 feet. Relatively low dead load when compared to a concrete superstructure makes this bridge type an asset in areas where foundation materials are poor.

2. Characteristics

   Construction details and forming are more difficult than for a steel plate girder. Shipping and erecting of large sections must be reviewed. Current cost information should be considered because of changing steel market conditions.

3. Depth/Span Ratios

   a. Constant depth

      Simple spans $1/22$

      Continuous spans $1/25$

   b. Variable depth

      At Center of span $1/40$

      At Intermediate pier $1/20$

      Note: Sloping webs are not used on box girders of variable depth.

H. Steel Truss

1. Application

   Used for simple spans up to 300 feet and for continuous spans up to 1,200 feet. Used where vertical clearance requirements dictate a shallow superstructure and long spans or where terrain dictates long spans and construction by cantilever method.

2. Characteristics

   Construction details are numerous and can be complex. Cantilever construction method can facilitate construction over inaccessible areas. Through trusses are discouraged because of the resulting restricted horizontal and vertical clearances for the roadway.

3. Depth/Span Ratios

   a. Simple spans $1/6$

   b. Continuous spans

      @ Center of span $1/18$

      @ Intermediate pier $1/9$

I. Segmental Concrete Box Girder

1. Application

   Used for continuous spans from 200 to 700 feet. Used where site dictates long spans and construction by cantilever method.

2. Characteristics

   Use of travelers for the form apparatus facilitates the cantilever construction method enabling long-span construction without falsework. Precast concrete segments may be used. Tight geometric control is required during construction to ensure proper alignment.

3. Depth/Span Ratios

   Variable depth

   At Center of span $1/50$

   At Intermediate pier $1/20$

J. Railroad Bridges

1. Use

   For railway over highway grade separations, most railroad companies prefer simple span steel construction. This is to simplify repair and reconstruction in the event of derailment or some other damage to the structure.

2. Characteristics

   The heavier loads of the railroad live load require deeper and stiffer members than for highway bridges. Through girders can be used to reduce overall structure depth if the railroad concurs. Piers should be normal to the railroad to eliminate skew loading effects.

3. Depth/Span Ratios

   Constant depth

Simple spans	$^1/_{12}$
Continuous two span	$^1/_{14}$
Continuous multi-span	$^1/_{15}$

K. Timber

1. Use

   Generally used for spans under 40 feet. Usually used for detour bridges and other temporary structures.

2. Characteristics

   Excellent for short-term duration as for a detour. Simple design and details.

3. Depth/Span Ratios

   Constant depth

Simple span – Timber beam	$^1/_{10}$
Simple span – Glulam beam	$^1/_{12}$
Continuous spans	$^1/_{14}$

L. Other

   Bridge types such as cable-stayed, suspension, arch, tied arch, and floating bridges have special and limited applications. The use of these bridge types is generally dictated by site conditions. Preliminary design studies will generally be done when these types of structures are considered.

2.4.2 Wall Types

Retaining walls, wingwalls, curtain walls, and tall closed abutment walls may be used where required to shorten spans or superstructure length or to reduce the width of approach fills. The process of selecting a type of retaining wall should economically satisfy structural, functional, and aesthetic requirements and other considerations relevant to a specific site. A detailed listing of the common wall types and their characteristics can be found in Chapter 8 of the *Bridge Design Manual.*

2.5 Aesthetic Considerations

2.5.1 General Visual Impact

A bridge has a strong visual impact in any landscape. Steps must be taken to assure that even the most basic structure will complement rather than detract from it's surroundings. The EIS and bridge site data submitted by the Region should each contain a discussion on the aesthetic importance of the project site. This commentary, together with submitted video and photographs, will help the designer determine the appropriate structure type.

The State Bridge and Structures Architect should be contacted early in the preliminary bridge plan process for input on aesthetics. Normally, a visit to the bridge site with the State Bridge and Structures Architect and Region design personnel should be made.

Aesthetics is a very subjective element that must be factored into the design process in the otherwise very quantitative field of structural engineering. Bridges that are well proportioned structurally using the least material possible are generally well proportioned. However, the details such as pier walls, columns, and crossbeams require special attention to ensure a structure that will enhance the general vicinity.

For large projects incorporating several to many bridges and retaining walls, an architectural theme is frequently developed to bring consistency in structure type, details, and architectural appointments. The preliminary plan designer shall work with the State Bridge and Structures Architect to implement the theme.

2.5.2 End Piers

A. Wingwalls

The size and exposure of the wingwall at the end pier should balance, visually, with the depth and type of superstructure used. For example, a prestressed girder structure fits best visually with a 15-foot wingwall (or curtain wall/retaining wall). However, there are instances where a 20-foot wingwall (or curtain wall/retaining wall) may be used with a prestressed girder (maximizing a span in a remote area, for example or with deep girders where they are proportionally better in appearance). The use of a 20-foot wingwall shall be approved by the Bridge Design Engineer and the State Bridge and Structures Architect.

It is less expensive for bridges of greater than 40 feet of overall width to be designed with wingwalls (or curtain wall/retaining wall) than to use a longer superstructure.

B. Retaining Walls

For structures at sites where profile, right of way, and alignment dictate the use of high exposed wall-type abutments for the end piers, retaining walls that flank the approach roadway can be used to retain the roadway fill and reduce the overall structure length. Stepped walls are often used to break up the height, and allow for landscape planting. A curtain wall runs between the bridge abutment and the heel of the abutment footing. In this way, the joint in the retaining wall stem can coincide with the joint between the abutment footing and the retaining wall footing. This simplifies design and provides a convenient breaking point between design responsibilities if the retaining walls happen to be the responsibility of the Region. The length shown for the curtain wall dimension is an estimated dimension based on experience and preliminary foundation assumptions. It can be revised under design to satisfy the intent of having the wall joint coincide with the end of the abutment footing.

C. Slope Protection

The Region is responsible for making initial recommendations regarding slope protection. It should be compatible with the site and should match what has been used at other bridges in the vicinity. The type selected shall be shown on the Preliminary Plan. It shall be noted on the “Not Included in Bridge Quantities” list.

2.5.3 Intermediate Piers

The size, shape, and spacing of the intermediate pier elements must satisfy two criteria. They must be correctly sized and detailed to efficiently handle the structural loads required by the design and shaped to enhance the aesthetics of the structure.

The primary view of the pier must be considered. For structures that cross over another roadway, the primary view will be a section normal to the roadway. This may not always be the same view as shown on the Preliminary Plan as with a skewed structure, for example. This primary view should be the focus of the aesthetic review.

Tapers and flares on columns should be kept simple and structurally functional. Fabrication and constructability of the formwork of the pier must be kept in mind. Crossbeam ends should be carefully reviewed. Skewed bridges and bridges with steep profile grades or those in sharp vertical curves will require special attention to detail.

Column spacing should not be so small as to create a cluttered look. Column spacing should be proportioned to maintain a reasonable crossbeam span balance.

2.5.4 Barrier and Wall Surface Treatments

A. Plain Surface Finish

This finish will normally be used on structures that do not have a high degree of visibility or where existing conditions warrant. A bridge in a remote area or a bridge among several existing bridges all having a plain finish would be examples.

B. Fractured Fin Finish

This finish is the most common and an easy way to add a decorative texture to a structure. Variations on this type of finish can be used for special cases. The specific areas to receive this finish should be reviewed with the State Bridge and Structures Architect.

C. Pigmented Sealer

The use of a pigmented sealer can also be an aesthetic enhancement. The particular hue can be selected to blend with the surrounding terrain. Most commonly, this would be considered in urban areas. The selection should be reviewed with the Bridge Architect and the Region.

D. Architectural Details

Rustication grooves, relief panels, pilasters, and decorative finishes may visually improve appearance at transitions between different structure types such as cast-in-place abutments to structural earth retaining walls. Contact the State Bridge and Structures Architect for guidance.

2.5.5 Superstructure

The horizontal elements of the bridge are perhaps the strongest features. The sizing of the structure depth based on the span/depth ratios in Section 2.4.1, will generally produce a balanced relationship.

Haunches or rounding of girders at the piers can enhance the structure's appearance. The use of such features should be kept within reason considering fabrication of materials and construction of formwork. The amount of haunch should be carefully reviewed for overall balance from the primary viewing perspective. Haunches are not limited to cast-in-place superstructures, but may be used in special cases on precast, prestressed I girders. They require job-specific forms which increase cost, and standard design software is not directly applicable.

The slab overhang dimension should approach that used for the structure depth. This dimension should be balanced between what looks good for aesthetics and what is possible with a reasonable slab thickness and reinforcement.

For box girders, the exterior webs can be sloped, but vertical webs are preferred. The amount of slope should not exceed l½: l for structural reasons, and should be limited to 4:1 if sloped webs are desired. Sloped webs should only be used in locations of high aesthetic impact.

2.6 Miscellaneous

2.6.1 Structure Costs

See Section 12.3 for preparing cost estimates for preliminary bridge design.

2.6.2 Handling and Shipping Precast Members and Steel Beams

Bridges utilizing precast concrete beams or steel beams need to have their access routes checked and sites reviewed to be certain that the beams can be transported to the site. It must also be determined that they can be erected once they reach the site.

Both the size and the weight of the beams must be checked. Likely routes to the site must be adequate to handle the truck and trailer hauling the beams. Avoid narrow roads with sharp turns, steep grades, and/or load-rated bridges, which may prevent the beams from reaching the site. The Bridge Preservation Office should be consulted for limitations on hauling lengths and weights.

Generally 200 kips is the maximum weight of a girder that may be hauled by truck. When the weight of a prestressed concrete girder cast in one piece exceeds 160 kips, it may be required to include a post-tensioned 2-piece option detailed in the contract plans.

The site should be reviewed for adequate space for the contractor to set up the cranes and equipment necessary to pick up and place the girders. The reach and boom angle should be checked and should accommodate standard cranes.

2.6.3 Salvage of Materials

When a bridge is being replaced or widened, the material being removed should be reviewed for anything that WSDOT may want to salvage. Items such as aluminum rail, luminaire poles, sign structures, and steel beams should be identified for possible salvage. The Region should be asked if such items are to be salvaged since they will be responsible for storage and inventory of these items.

2.7 WSDOT Standard Highway Bridge

2.7.1 Design Elements

The following are standard design elements for bridges carrying highway traffic. They are meant to provide a generic base for consistent, clean looking bridges, and to reduce design and construction costs. Modification of some elements may be required, depending on site conditions. This should be determined on a case-by-case basis during the preliminary plan stage of the design process.

A. General

Fractured Fin Finish shall be used on the exterior face of the traffic barrier. All other surfaces shall be Plain Surface Finish.

Exposed faces of wingwalls, columns, and abutments shall be vertical. The exterior face of the traffic barrier and the end of the intermediate pier crossbeam and diaphragm shall have a 1:12 backslope.

B. Substructure

End piers use the following details:

15 feet wingwalls with prestressed girders up to 74 inches in depth or a combination of curtain wall/retaining walls.

Stub abutment wall with vertical face. Footing elevation, pile type (if required), and setback dimension are determined from recommendations in the Materials Laboratory Geotechnical Services Branch Geotechnical Report.

Intermediate piers use the following details:

“Semi-raised” Crossbeams: The crossbeam below the girders is designed for the girder and slab dead load, and construction loads. The crossbeam and the diaphragm together are designed for all live loads and composite dead loads. The minimum depth of the crossbeam shall be 3 feet.

“Raised” Crossbeams: The crossbeam is at the same level as the girders are designed for all dead and live loads. “Raised” crossbeams are only used in conjunction with Prestressed Concrete Tub Girders.

Round Columns: Columns shall be 3 feet to 6 feet in diameter. Dimensions are constant full height with no tapers. Bridges with roadway widths of 40 feet or less will generally be single column piers. Bridges with roadway widths of greater the 40 feet shall have two or more columns, following the criteria established in Section 2.3.1.H. Oval or rectangular column may be used if required for structural performance or bridge visual.

C. Superstructure

Concrete Slab: 7½ inch minimum thickness, with the top and bottom mat being epoxy coated steel reinforcing bars.

Prestressed Girders: Girder spacing will vary depending on roadway width and span length. The slab overhang dimension is approximately half of the girder spacing. Girder spacing typically ranges between 6 feet and 12 feet.

Intermediate Diaphragms: Locate at the midspan for girders up to 80 feet long. Locate at third points for girders between 80 feet and 150 feet long and at quarter points for spans over 150 feet.

End Diaphragms: “End Wall on Girder” type.

Traffic Barrier: “F-shape” or Single-sloped barrier.

Fixed Diaphragm at Inter. Piers: Full or partial width of crossbeam between girders and outside of the exterior girders.

Hinged Diaphragm at Inter. Piers: Partial width of crossbeam between girders. Sloped curtain panel full width of crossbeam outside of exterior girders, fixed to ends of crossbeam.

BP Rail: 3 feet 6 inches overall height for pedestrian traffic. 4 feet 6 inches overall height for bicycle traffic.

Sidewalk: 6-inch height at curb line. Transverse slope of -0.02 feet per foot towards the curb line.

Sidewalk barrier: Inside face is vertical. Outside face slopes 1:12 outward.

D. Examples

Appendices 2.3-A2-1 and 2.7-A1-1 detail the standard design elements of a standard highway bridge.

The following bridges are good examples of a standard highway bridge. However, they do have some modifications to the standard.

SR 17 Undercrossing 395/110	Contract 3785
Mullenix Road Overcrossing 16/203E&W	Contract 4143

2.7.2 Detailing the Preliminary Plan

The Bridge Preliminary Plan is used and reviewed by the Bridge and Structures Office or consultant who will do the structural design, Region designers and managers, Geotechnical engineers, Hydraulics engineers, Program managers, FHWA engineers and local agency designers and managers. It sometimes is used in public presentation of projects. With such visibility it is important that it's detailing is clear, complete, professional, and attractive. The designer, detailer, and checker shall strive for completeness and consistency in information, layout, line style, and fonts. Appendix B contains examples of Preliminary Plans following time-proven format that may be helpful. See also Chapter 11, Detailing Practice.

Typical sheet layout is as follows:

1. Plan and Elevation views. (This sheet ultimately becomes the Layout sheet of the design plan set)
2. Typical Section including details of stage construction.

   Superelevation diagrams, tables of existing elevations, Notes to Region, and other miscellaneous details as required shall go on Sheet 2, 3, or 4, as many as are required. See also the Preliminary Plan Checklist for details, dimensions, and notes typically required. The completed plan sheets shall be reviewed for consistency by the Preliminary Plans Detailing Specialist.

2.99 Bibliography

1. Federal Highway Administration (FHWA) publication *Federal Aid Highway Program Manual*.

   FHWA Order 5520.1 (dated December 24, 1990) contains the criteria pertaining to Type, Size, and Location studies.

   Volume 6, Chapter 6, Section 2, Subsection 1, Attachment 1 (Transmittal 425) contains the criteria pertaining to railroad undercrossings and overcrossings.

2. Washington Utilities and Transportation Commission *Clearance Rules and Regulations Governing Common Carrier Railroads.*

3. American Railway Engineering and Maintenance Association (AREMA) *Manual for Railroad Engineering*. Note: This manual is used as the basic design and geometric criteria by all railroads. Use these criteria unless superseded by FHWA or WSDOT criteria.

4. Washington State Department of Transportation (WSDOT) *Design Manual* (M 22-01).

5. *Local Agency Guidelines* (M 36-63).

6. American Association of State Highway and Transportation Officials *AASHTO LRFD Bridge Design Specification.*

7. The Union Pacific Railroad *"Guidelines for Design of Highway Separation Structures over Railroad (Overhead Grade Separation)"*

Contents

Chapter 5 Concrete Structures

For sections with bonded tendons:

$$c = \frac{A_{ps} f_{pu} + A_s f_y - A'_s f'_y \text{ -0.85 } f'_c (b - b_w) h_f}{0.85 f'_c \beta_1 b_w + k A_{ps} \dfrac{f_{pu}}{d_p}} \quad (5.2.9\text{-}1)$$

For sections with unbonded tendons:

$$c = \frac{A_{ps} f_{ps} + A_s f_y - A'_s f'_y \text{ -0.85 } f'_c (b - b_w) h_f}{0.85 f'_c \beta_1 b_w} \quad (5.2.9\text{-}2)$$

Consequently, the nominal flexural resistance of LRFD Equation 5.7.3.2.2-1 shall be modified by deleting β_1 from the last term as shown below:

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_y\left(d_s - \frac{a}{2}\right) - A'_s f'_y\left(d'_s - \frac{a}{2}\right) + 0.85 f'_c (b - b_w) h_f\left(\frac{a}{2} - \frac{h_f}{2}\right) \quad (5.2.9\text{-}3)$$

C. Nominal Flexural Resistance

1. Theoretical Background

These provisions could be considered a philosophical change to traditional flexural resistance calculations of reinforced and prestressed concrete members. In these provisions sections are considered either tension –controlled, transition or - compression controlled. Classifying sections as tension-controlled, transition or compression-controlled, and linearly varying the resistance factor in the transition zone between values for the two extremes, provides a rational approach for determining ϕ and limiting the capacity of over-reinforced sections.

Sections are compression-controlled when the net tensile strain in the extreme tension steel is equal to or less than the compression-controlled strain limit at the time the concrete in compression reaches its assumed strain limit of 0.003. The compression-controlled strain limit is the net tensile strain in the reinforcement at balanced strain conditions. For Grade 60 reinforcement, and for all prestressed reinforcement, the compression-controlled strain limit may be set equal to 0.002.

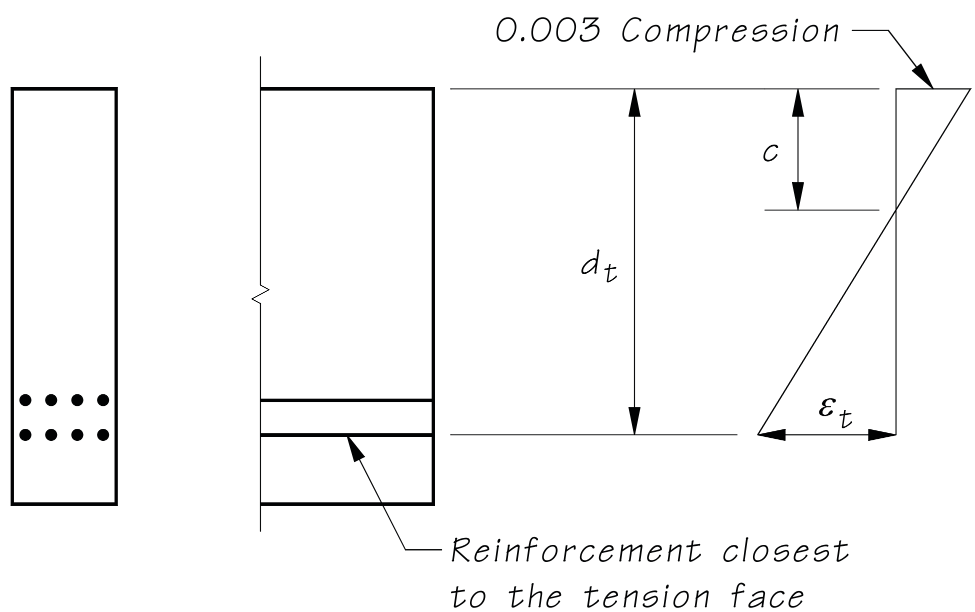
Sections are tension-controlled when the net tensile strain in the extreme tension steel is equal to or greater than 0.005 just as the concrete in compression reaches its assumed strain limit of 0.003. Sections with net tensile strain in the extreme tension steel between the compression-controlled strain limit and 0.005 constitute a transition region between compression-controlled and tension-controlled sections.

Balanced strain conditions exist at a cross section when tension reinforcement reaches the strain corresponding to its specified yield strength f_y just as the concrete in compression reaches its assumed ultimate strain of 0.003.

For beams with compression reinforcement, or T-beams, the effects of compression reinforcement and flanges are automatically accounted for in the computation of net tensile strain ε_t. The use of compression reinforcement in conjunction with additional tension reinforcement is permitted to increase the strength of flexural members.

2. Nominal Flexural Resistance

The nominal flexural resistance of a member is reached when the strain in the extreme compression fiber reaches the assumed strain limit of 0.003. The net tensile strain εt is the tensile strain in the extreme tension steel at nominal strength, exclusive of strains due to prestress, creep, shrinkage and temperature. The net tensile strain in the extreme tension steel is determined from a linear strain distribution at nominal strength, as shown in Figure 5.2.9-1, using similar triangles.



Strain distribution and net tensile strain
Figure 5.2.9-1

When the net tensile strain in the extreme tension steel is sufficiently large (equal to or greater than 0.005), the section is defined as tension-controlled where ample warning of failure with excessive deflection and cracking may be expected. When the net tensile strain in the extreme tension steel is small (less than or equal to the compression-controlled strain limit), a brittle failure condition may be expected, with little warning of impending failure. Flexural members are usually tension-controlled, while compression members are usually compression-controlled. Some sections, such as those with small axial load and large bending moment, will have net tensile strain in the extreme tension steel between the above limits. These sections are in a transition region between compression- and tension-controlled sections.

3. Resistance Factors

The resistance factors for tension-controlled and compression-controlled sections, and for intermediate cases in the transition region shall be taken as in Table 5.2.9-1:

		Precast Members	Cast-in-Place Members
Conventional Construction (other than Segmentally constructed Bridges)	Mild Reinforcement	1.0	0.9
	Prestressed	1.0	0.9
	Spliced Girders	0.95	

Flexural Resistance Factor for Tension-Controlled Concrete Members
Table 5.2.9-1

For compression-controlled members, regardless of the method of construction, the flexural resistance factor will continue to be taken as 0.75.

For sections in which the net tensile strain in the extreme tension steel at nominal strength is between the limits for compression-controlled and tension-controlled sections, ϕ may be linearly increased from 0.75 to that for tension-controlled sections as the net tensile strain in the extreme tension steel increases from the compression-controlled strain limit to 0.005.

For members in the transition zone between tension-controlled and compression-controlled, the flexural resistance factor shall be taken as follows:

For precast members:

$$0.75 \le \varphi = 0.583 + 0.25\left(\frac{d_t}{c} - 1\right) \le 1.0 \qquad (5.2.9\text{-}4)$$

For cast-in-place members:

$$0.75 \le \varphi = 0.650 + 0.15\left(\frac{d_t}{c} - 1\right) \le 0.9 \qquad (5.2.9\text{-}5)$$

For precast spliced girders with cast-in-place closures:

$$0.75 \le \varphi = 0.616 + 0.20\left(\frac{d_t}{c} - 1\right) \le 0.95 \qquad (5.2.9\text{-}6)$$

where:

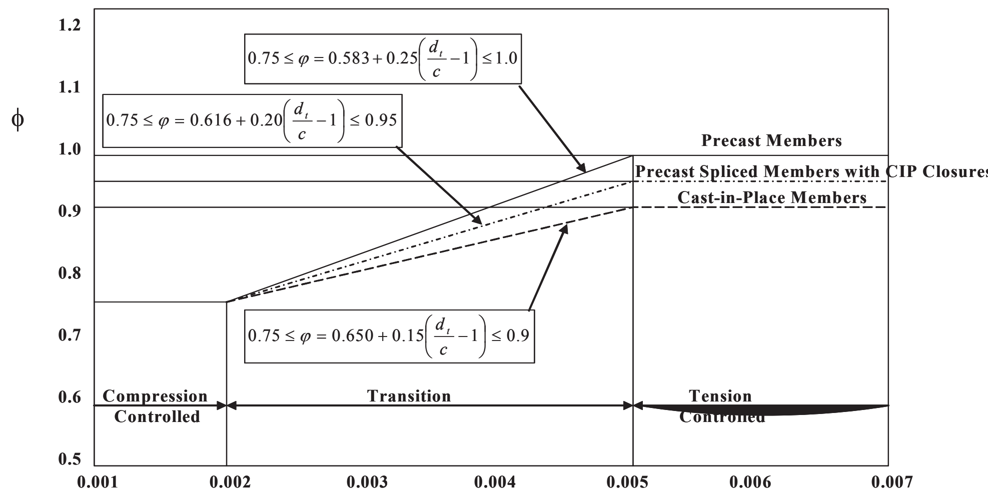
d_t = distance from extreme compression fiber to centroid of extreme tension steel

ε_t = net tensile strain in extreme tension steel at nominal strength

In applying the resistance factors for tension-controlled and compression-controlled sections, the axial tensions and compressions to be considered are those caused by external forces. Effects of prestressing forces are not included.

A lower ϕ-factor is used for compression-controlled sections than is used for tension-controlled sections because compression-controlled sections have less ductility, are more sensitive to variations in concrete strength, and generally occur in members that support larger loaded areas than members with tension-controlled sections.

For sections subjected to axial load with flexure, factored resistances are determined by multiplying both P_n and M_n by the appropriate single value of ϕ. Compression-controlled and tension-controlled sections are defined as those that have net tensile strain in the extreme tension steel at nominal strength less than or equal to the compression-controlled strain limit, and equal to or greater than 0.005, respectively. For sections with net tensile strain ε_t in the extreme tension steel at nominal strength between the above limits, the value of ϕ may be determined by linear interpolation, as shown in Figure 5.2.9-2.



Variation of ϕ with Net Tensile strain ε_t

Figure 5.2.9-2

D. Limit of Reinforcement

The LRFD Specifications do not handle maximum reinforcement limits for prestressed and non-prestressed flexural members in a consistent manner. While over-reinforced non-prestressed flexural members are not allowed, over-reinforced prestressed flexural members are allowed if sufficient ductility of the structure can be achieved.

LRFD specifications limit the tension reinforcement quantity to a maximum amount such that the ratio c/d_e did not exceed 0.42. Sections with $c/d_e > 0.42$ were considered over-reinforced. Over-reinforced nonprestressed members were not allowed, whereas prestressed and partially prestressed members with PPR greater than 50 percent were if "it is shown by analysis and experimentation that sufficient ductility of the structure can be achieved." No guidance was given for what "sufficient ductility" should be, and it was not clear what value of ϕ should be used for such over-reinforced members. These provisions eliminate this limit and unify the design of prestressed and nonprestressed tension- and compression-controlled members. The background and basis for these provisions are given in references 31, 32, 33 and 34.

E. Moment Redistribution

In lieu of more refined analysis, where bonded reinforcement is provided at the internal supports of continuous reinforced concrete beams, negative moments determined by elastic theory at strength limit states may be increased or decreased by not more than 1000 ε_t percent, with a maximum of 20 percent. Redistribution of negative moments shall be made only when ε_t is equal to or greater than 0.0075 at the section at which moment is reduced.

Unless unusual amounts of ductility are required, the 0.005 limits will provide ductile behavior for most designs. One condition where greater ductile behavior is required is in design for redistribution of moments in continuous members and frames. Since moment redistribution is dependent on adequate ductility in hinge regions, moment redistribution is limited to sections that have a net tensile strain of at least 0.0075.

Design example 12 in Appendix B illustrates the flexural strength calculations for Composite T-Beam

5.2.10 Shrinkage and Temperature Reinforcement

Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete exposed to daily temperature changes and in structural mass concrete. Temperature and shrinkage reinforcement shall ensure that the total reinforcement on the exposed surfaces in not less than that specified herein. Reinforcement for shrinkage and temperature may be in the form of bars, welded wire fabric or prestressing tendons.

For bar or welded wire fabric, the area or reinforcement per foot, on each face and in each direction shall satisfy:

$$A_s \geq \frac{1.30bh}{2(b+h)f_y} \qquad (5.2.10\text{-}1)$$

$$0.11 \leq A_s \leq 0.60 \qquad (5.2.10\text{-}2)$$

A_s = area of reinforcement in each direction and each face (in^2/ft)

b = least width of component section (in.)

h = least thickness of component section (in.)

b = specified yield strength of reinforcing bars ≤ 75 ksi

Where the least dimension varies along the length of wall, footing, or other component, multiple sections should be examined to represent the average condition at each section. Spacing is not the exceed:

- 3.0 times the component thickness, or 18.0 in.
- 12.0 in. for walls and footings over 18.0 in. thick
- 12.0 for other components over 36.0 in. thick

For components 6.0 in. or less in thickness the minimum steel specified may be placed in a single layer. Shrinkage and temperature steel is not required for:

- End face of walls 18 in. or less in thickness
- Side faces of footings 36 in. or less in thickness
- Faces of all other components, with smaller dimension less than or equal to 18.0 in.

If prestressing tendons are used as steel for shrinkage and temperature reinforcement, the tendons shall provide a minimum average compressive stress of 0.11 reinforcement, the tendons shall provide a minimum average compressive stress of 0.11 ksi on the gross concrete area through which a crack plane may extend, based on the effective prestress after losses. Spacing of tendons should not exceed either 72.0 in. or the distance specified in Article 5.10.3.3. Where the spacing is greater than 54.0 in., bonded reinforcement shall be provided between tendons, for a distance equal to the tendon spacing.

5.2.11 Minimum Reinforcement Requirement

Unless otherwise specified, at any section of a flexural component, the amount of tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r at least equal to:

- For nonprestressed flexural members, the amount of tensile reinforcement shall be adequate to develop 1.0 times the cracking moment, M_{cr}, and for prestressed flexural members, the amount of prestressed and nonprestressed tensile reinforcement shall be adequate to develop 1.33 times the cracking moment; where
- The cracking moment, M_{cr} is determined on the basis of elastic stress distribution and the modulus of rupture, f_r, of concrete as specified in LRFD Article 5.4.2.6. M_{cr} may be taken as:

$$M_{cr} = S_c\left(f_r + f_{cpe}\right) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) \qquad (5.2.11\text{-}1)$$

where:

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

M_{dnc}= total unfactored dead load moment acting on the monolithic or noncomposite section (kip-ft.)

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

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

Appropriate values for M_{dnc} and S_{nc} shall be used for any intermediate composite sections. Where the beams are design for the monolithic or noncomposite section to resister all loads, substitute S_{nc} for S_c in the above equation for the calculation of M_{cr}.

This provision shall be permitted to be waived for:

- Nonprestressed members with flexural strength at least 1.33 times the factored moment required by the applicable strength load combinations specified in LRFD Table 3.4.1-1, and
- Prestressed members with flexural strength at least 2.0 times the factored moment required by the applicable strength load combinations specified in LRFD Table 3.4.1.-1

8. Location of holes and shear keys for intermediate and end diaphragms shall be verified per contract plans.
9. Location and size of bearing recesses shall be verified per contract plans.
10. Saw tooth at girder ends shall be verified per contract plans.
11. Location and size of lifting loops or lifting bars shall be verified per contract plans.
12. All horizontal and vertical reinforcement shall be verified per contract plans.
13. Girder length and end skew shall be verified per contract plans.

5.7 Roadway Slab

The following information is intended to provide guidance for slab thickness and transverse and longitudinal reinforcement of roadway slab. Information on deck deterioration prevention systems is section 5.7.4.

5.7.1 Roadway Slab Requirements

A. Slab Thickness

Slab thickness for prestressed girder bridges shall be taken as shown in Table 5.7.2-2.

The minimum slab thickness is established in order to ensure that overloads on the bridge will not result in premature slab cracking.

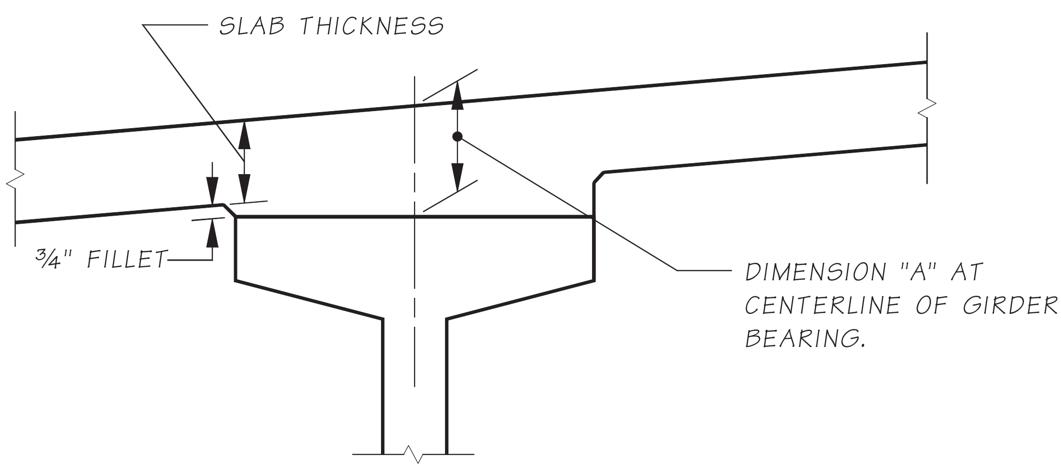
B. Computation of Slab Strength

The thickness for usual slabs are shown in Figure 5.7.1-1 and Figure 5.7.1-2. The slab design span and thickness are defined in Table 5.7.2-2

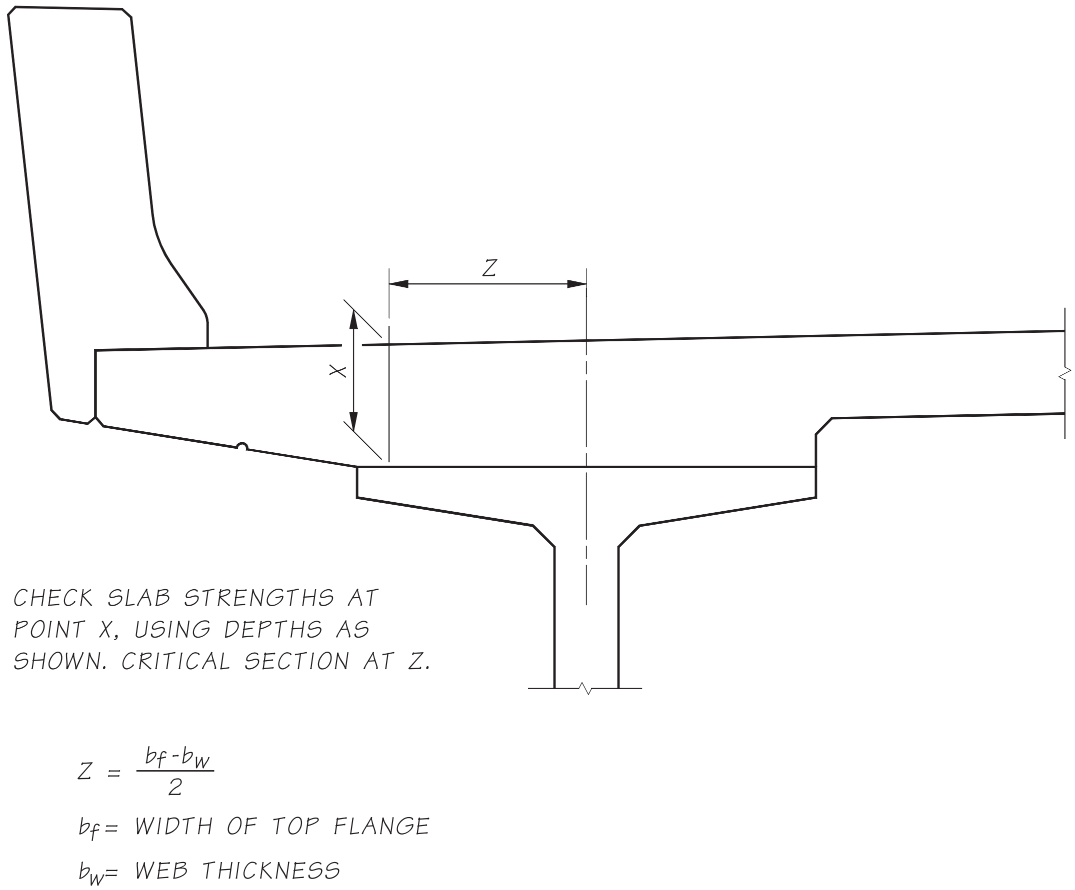
The thickness of the slab and reinforcement in the area of the cantilever may be governed by traffic barrier loading. Wheel loads plus dead load shall be resisted by the sections shown in Figure 5.7.1-2.

Cantilever loads may govern the slab thickness just inside the exterior girder as shown by “Z” in Figure 5.7.1-2.

Design of the cantilever is normally based on the expected depth of slab at centerline of girder span. This is less than the dimensions at the girder ends (somtimes).



Depths for Slab Design at Centerline of Girder Span
Figure 5.7.1-1



Depths for Slab Design at Deck Overhang
Figure 5.7.1-2

C. Computation of "A" Dimension

The distance from the top of the slab to the top of the girder at centerline bearing is represented by the "A" Dimension. It is calculated in accordance with the guidance of Appendix B. This ensures that adequate allowance will be made for excess camber, transverse deck slopes, vertical and horizontal curvatures. Ideally the section at centerline of span will have the final geometry shown in Figure 5.7.1-1. Where temporary prestress strands at top of girder are used to control the girder stresses due to shipping and handling, the "A" dimension must be adjusted accordingly.

The note in left margin of the Layout Sheet should read: "A" Dimen. = X" (not for design).

5.7.2 Slab Reinforcement

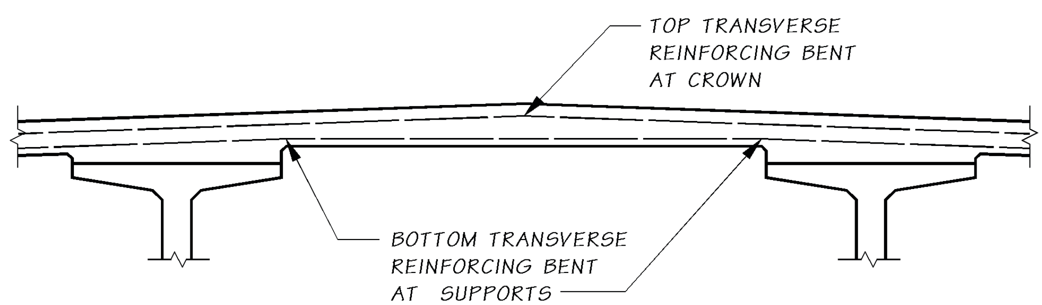
A. Transverse Reinforcement

The size and spacing of transverse reinforcement may be governed by interior slab span design, cantilever design, or the requirements of traffic barrier load. Where traffic barrier load governs, short hooked bars may be added at the slab edge to increase the reinforcement available in that area. Top transverse reinforcement is always hooked at the slab edge unless a traffic barrier is not used. Top transverse reinforcement is preferably spliced at some point between girders in order to allow the clearance of the hooks to the slab edge forms to be properly adjusted in the field. Usually, the slab edge hooks will need to be tilted in order to place them. On larger bars, the clearance for the longitudinal bar through the hooks should be checked. Bottom transverse slab reinforcement is normally carried far enough to splice with the traffic barrier main reinforcement. Appendices 5.3-A5 through 5.3-A8 can be used to aid in selection of bar size and spacing.

For skewed spans, the transverse bars are placed normal to bridge centerline and the areas near the expansion joints and bridge ends are reinforced by partial length bars. For raised crossbeam bridges, the bottom transverse slab reinforcement is discontinued at the crossbeam.

The spacing of bars over the crossbeam must be detailed to be open enough to allow concrete to be poured into the crossbeam. For typical requirements, see Section 5.3.3.D.

For slabs with a crowned roadway, the bottom surface and rebar of the slab should be flat, as shown in Figure 5.7.2-1.



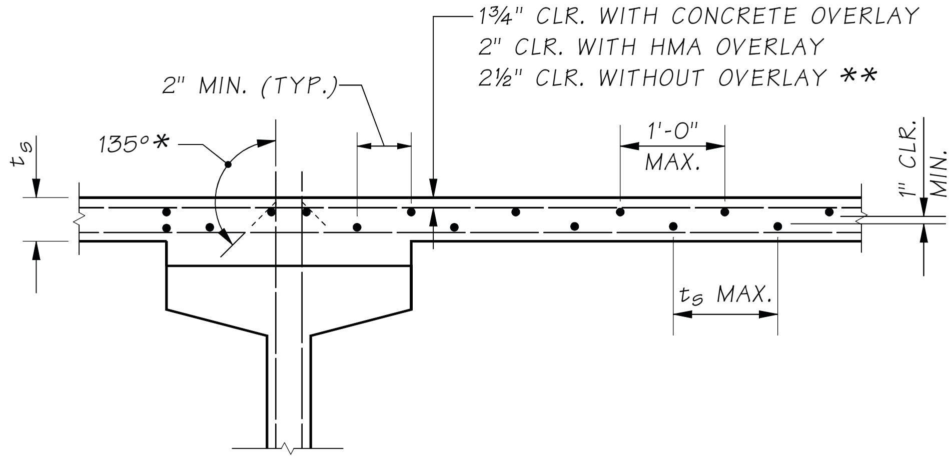
Bottom of Top Slab at Crown Point
Figure 5.7.2-1

B. Longitudinal Reinforcement

This section discusses reinforcement requirements for resistance of longitudinal moments in continuous multi-span precast girder bridges and is limited to reinforcement in the top slab since capacity for resisting positive moment is provided by the prestressing of the girders.

1. Simple Spans

For simple span bridges, longitudinal slab reinforcement is not required to resist negative moments and therefore the reinforcement requirements are nominal. Figure 5.7.2-2 defines longitudinal reinforcement requirements for these slabs. The bottom longitudinal reinforcement is defined by AASHTO requirements for distribution reinforcement. The top longitudinal reinforcement is based on current office practice. The requirements of Distribution of Flexural Reinforcement do not apply to these bars.



* TOP MAT REBARS SHALL BE EPOXY COATED. STIRRUPS NEED NOT BE EPOXY COATED AND SHALL BE BENT AT 135°.

** THESE CLEARANCE REQUIREMENTS APPLY IRRESPECTIVE OF WHETHER OR NOT THE REBARS IN THE TOP MAT ARE EPOXY COATED.

Nominal Longitudinal Slab Reinforcement
Figure 5.7.2-2

2. Continuous Spans

   Longitudinal reinforcement of continuous spans at intermediate support is dominated by the moment requirement. Where these bars are cut off, they are lapped by the nominal top longitudinal reinforcement described in Subsection 5.7.2D. Minimum sub thickness is shown in Table 5.7.2-1.

C. Distribution of Flexural Reinforcement

The provision of LRFD Section 5.7.3.4 for class 2 exposure condition shall be satisfies.

1. Prestressed Girders Designed as Simple Spans

   For bridges designed using the "Prestressed Girder Design" program, "distribution reinforcement" need not be added to the area of steel required to resist the negative moments. The bars in the bottom layer, however, shall provide an area not less than that required for distribution reinforcement.

2. Other Prestressed Girder Bridges

   On bridges where the effect of continuity is taken into account to reduce moments for girder design, additional longitudinal steel shall be provided as "distribution reinforcement." The sum of the areas in both layers of longitudinal bars shall be equal to the area required to resist negative moments plus the area required by the AASHTO specification for "distribution reinforcement." Equal area of reinforcement shall be used in the top and bottom layers throughout the negative moment region. The total area of steel required in the bottom longitudinal layer shall not be less than that required for "distribution reinforcement."

The minimum clearance between top and bottom bars should be 1-inch. Table 5.7.2-1 shows required slab thickness for various bar combinations. Table 5.7.2-2 shows the minimum slab thickness for different types of prestresses girders.

Minimum Slab Thickness = 7 Inches	Slab Thickness (Inches)		
	Transverse Bar		
Longitudinal Bar	#5	#6	#7
#4	7½	--	--
5	7½	7½	7¾
6	7½	7¾	8
7	7¾	8	8¼
8	8	8½	8¾
9	8½	8¾	9
10	8¾	--	--

Note: Deduct ½-inch from slab thickness shown in table when asphalt overlay is used and 1 inch when concrete overlay is used. However, the minimum slab thickness shall be 7 inches when overlay is used.

Minimum Slab Thickness for Various Bar Sizes (Slab Without Overlay)

Table 5.7.2-1

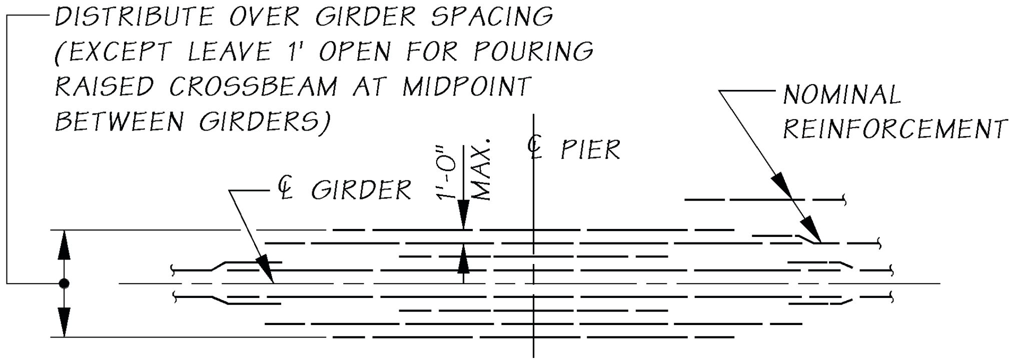
Girder Type	Web Spacing ft	Width of Top Flange ft	Web Thickness in	Effective Girder Spacing ft	Minimum Slab Thickness in	Design Minimum Slab Thickness in
W42G	5	1.250	6	4.125	7.50	7.50
	6	1.250	6	5.125	7.50	7.50
	7	1.250	6	6.125	7.50	7.50
	8	1.250	6	7.125	7.50	7.50
	9	1.250	6	8.125	7.50	7.50
	10	1.250	6	9.125	7.65	7.75
	11	1.250	6	10.125	8.05	8.25
	12	1.250	6	11.125	8.45	8.50
W50G	5	1.667	6	3.917	7.50	7.50
	6	1.667	6	4.917	7.50	7.50
	7	1.667	6	5.917	7.50	7.50
	8	1.667	6	6.917	7.50	7.50
	9	1.667	6	7.917	7.50	7.50
	10	1.667	6	8.917	7.57	7.75
	11	1.667	6	9.917	7.97	8.00
	12	1.667	6	10.917	8.37	8.50
W58G	5	2.083	6	3.709	7.50	7.50
	6	2.083	6	4.709	7.50	7.50
	7	2.083	6	5.709	7.50	7.50
	8	2.083	6	6.709	7.50	7.50
	9	2.083	6	7.709	7.50	7.50
	10	2.083	6	8.709	7.50	7.50
	11	2.083	6	9.709	7.88	8.00
	12	2.083	6	10.709	8.28	8.50
W74G	5	3.583	6	2.958	7.50	7.50
	6	3.583	6	3.958	7.50	7.50
	7	3.583	6	4.958	7.50	7.50
	8	3.583	6	5.958	7.50	7.50
	9	3.583	6	6.958	7.50	7.50
	10	3.583	6	7.958	7.50	7.50
	11	3.583	6	8.958	7.58	7.75
	12	3.583	6	9.958	7.98	8.00

WF42G, WF50G, WF58G, WF74G, W83G, & W95G	6	4.083	6.125	3.703	7.50	7.50
	7	4.083	6.125	4.703	7.50	7.50
	8	4.083	6.125	5.703	7.50	7.50
	9	4.083	6.125	6.703	7.50	7.50
	10	4.083	6.125	7.703	7.50	7.50
	11	4.083	6.125	8.703	7.50	7.50
	12	4.083	6.125	9.703	7.88	8.00
W32BTG, W38BTG, & W62BTG	5	4.083	6	2.708	7.50	7.50
	6	4.083	6	3.708	7.50	7.50
	7	4.083	6	4.708	7.50	7.50
	8	4.083	6	5.708	7.50	7.50
	9	4.083	6	6.708	7.50	7.50
	10	4.083	6	7.708	7.50	7.50
	11	4.083	6	8.708	7.50	7.50
	12	4.083	6	9.708	7.88	8.00
WF74PTG, W83PTG, & W95PTG	6	4.229	7.875	3.557	7.50	7.50
	8	4.229	7.875	5.557	7.50	7.50
	10	4.229	7.875	7.557	7.50	7.50
	12	4.229	7.875	9.557	7.82	8.00
	14	4.229	7.875	11.557	8.62	8.75
UF60G, UF72G, & UF84G	6	1.256	7	5.080	7.50	7.50
	7	1.256	7	6.080	7.50	7.50
	8	1.256	7	7.080	7.50	7.50
	9	1.256	7	8.080	7.50	7.50
	10	1.256	7	9.080	7.63	7.75
	11	1.256	7	10.080	8.03	8.25
	12	1.256	7	11.080	8.43	8.50

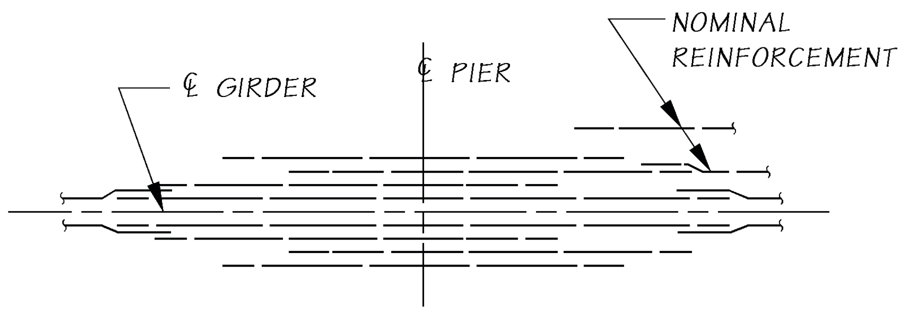
Minimum Slab Thickness for Prestressed Girder Bridges
Table 5.7.2-2

D. Bar Patterns

Figure 5.7.2-3 shows two typical top longitudinal reinforcing bar patterns. Care must be taken that bar lengths conform to the requirements of Table 5.1.2-2. Note that the reinforcement is distributed over a width equal to the girder spacing according to office practice and does not conform to AASHTO LRFD Specifications Section 9.7.3.2.



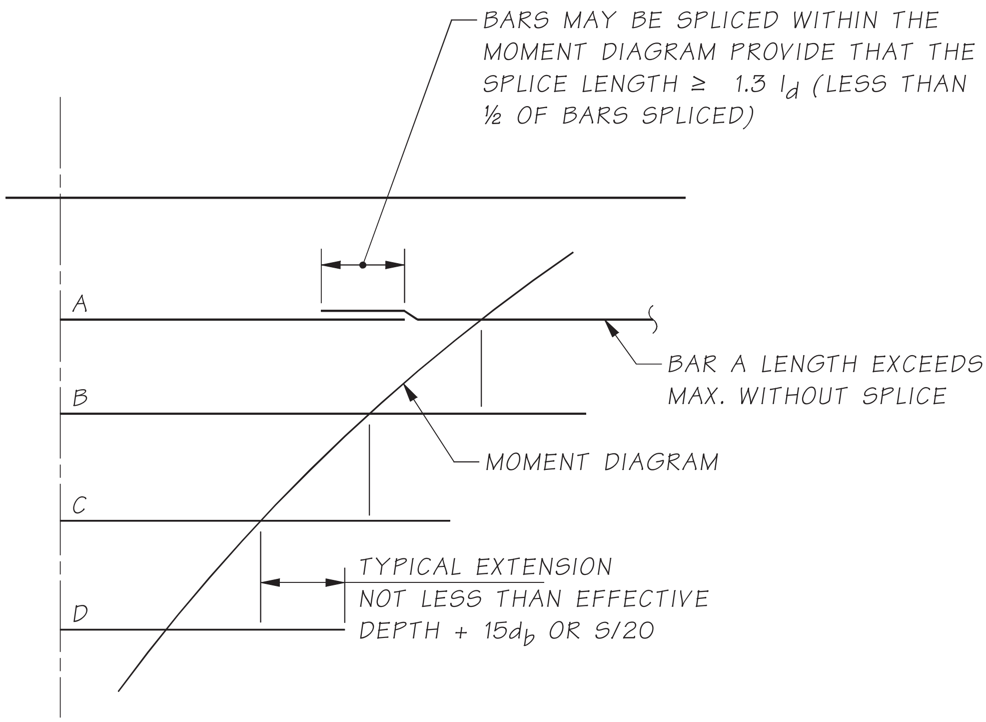
SYMMETRICAL BAR PATTERN
FOR BAR LENGTH < 60 FEET



STAGGERED BAR PATTERN
FOR BAR LENGTH > 60 FEET

Staggered Bar Pattern
Figure 5.7.2-3

The symmetrical bar pattern shown should normally not be used when required bar lengths exceed 60 feet. If the staggered bar pattern will not result in bar lengths within the limits specified in Table 5.1.2-2, the method shown in Figure 5.7.2-4 may be used to provide an adequate splice. All bars shall be extended by their development length beyond the point where the bar is required.



Bar Splice Within Moment Envelope
Figure 5.7.2-4

In all bar patterns, the reinforcement shall be well distributed between webs. Where this cannot be done without exceeding the 1-foot 0-inch maximum spacing requirement, the nominal longitudinal bars may be extended through to provide the 1-foot 0-inch maximum.

Normally, no more than 20 percent of the main reinforcing bars shall be cut off at one point. Where limiting this value to 20 percent leads to severe restrictions on the reinforcement pattern, an increase in figure may be considered. Two main reinforcement bars shall be carried through the positive moment area as stirrup hangers.

E. Recommendations for Concrete Deck Slab Detailing

These recommendations are primarily for beam-slab bridges with main reinforcement perpendicular to traffic.

- The minimum slab thickness including 0.5 in. wearing surface shall be 7.5 in. for concrete bridges 8.0 in. for steel girder bridges, and 8.5 in. for concrete decks with S-I-P deck panels.
- Minimum cover over the top layer of reinforcements shall be 2.5 in. including 0.5 in. wearing surface. The minimum cover over the bottom layer reinforcement shall be 1.0 in..
- Maximum bare size of #5 is preferred for all longitudinal and transverse reinforcements in deck slab except maximum bar size of #7 may be used for longitudinal reinforcements at intermediate piers.
- The minimum amount of reinforcement in each direction shall be 0.18 in.^{2}/ft for the top layer and 0.27 in.^{2}/ft for the bottom layer. The amount of longitudinal reinforcement in the bottom of slabs shall not be less than $220/\sqrt{S} \leq 67$ percent moment as specified in AASHTO LRFD 9.7.3.2.

- Top and bottom reinforcement in longitudinal direction of deck slab shall be staggered to allow better flow of concrete between the reinforcing bars.
- The maximum bar spacing in transverse and longitudinal directions for the top mat, and transverse direction of the bottom mat shall not exceed 12 inches. The maximum bar spacing for bottom longitudinal within the effective length, as specified in LRFD article 9.7.2.3, shall not exceed the deck thickness.
- For bridges with skew angle exceeding 25 degrees, the amount of reinforcement in both primary and secondary direction shall be increased in the end zones.
- The construction joint with roughened surface in the slab at the intermediate pier diaphragm shall be specified instead of construction joint with shear key.
- Both, top and bottom layer reinforcement shall be considered when designing for negative moment at the intermediate piers.
- Reduce lap splices if possible. Use staggered lap splices for both top and bottom in longitudinal and transverse directions

5.7.3 Stay-In-Place Deck Panels

A. General

The use of stay-in-place (S-I-P) deck panels for bridge decks shall be investigated at the preliminary design stage. A minimum slab thickness of 8 ½”, including 3 ½” inch precast deck panel and 5.0 inch cast-in-place concrete topping, shall be specified for design if S-I-P deck panels are considered for the bridge deck. The acceptance evaluation will consider such items as extra weight for seismic design and the resulting substructure impacts.

The composite deck system consisting of precast prestressed concrete deck panels with a cast-in-place topping has advantages in minimizing traffic disruption, speeding up construction and solving constructability issues on certain projects. Contractors, in most cases, prefer this composite deck panel system for bridge decks in traffic congested areas and other specific cases.

Composite deck slab system consisting of S-I-P precast concrete deck panels combined with cast-in-place concrete topping may be used on WSDOT bridges upon Region’s request and Bridge and Structures Office approval. Details for S-I-P deck panels are shown in Appendix 5.6-A18-1.

B. Design Criteria

The design of S-I-P deck panels follows the AASHTO LRFD Bridge Design Specifications and the PCI Bridge Design Manual. The design philosophy of S-I-P deck panels is identical to simple span prestressed girders. They are designed for Service Limit State and checked for Strength Limit State. The precast panels support the dead load of deck panels and C-I-P topping, and the composite S-I-P deck panel and C-I-P cross-section resists the live load. The tensile stress at the bottom of the panel is limited to zero per WSDOT design practice.

C. Limitation on S-I-P Deck Panels

The conventional full-depth cast-in-place deck slab continuous to be preferred by the Bridge and Structures Office for most applications. However, WSDOT allows the use of S-I-P Deck Panels upon Regions request and Bridge & Structures Office approval with the following limitations:

1. S-I-P Deck Panels shall not be used in Negative moment region of continuous conventionally reinforced bridges. S-I-P Deck Panels may be used in post-tensioned continuous bridges.

2. Bridge widening. S-I-P Deck Panels are not allowed in the bay adjacent to the existing structure because it is difficult to set the panels properly on the existing structure, and the requirement for C-I-P closure. S-I-P Deck Panels can be used on the other girders when the widening involves multiple girders.
3. Phased construction. S-I-P Deck Panels are not allowed in the bay adjacent to the previously placed deck because of the requirement for C-I-P closure.
4. Prestressed girders with narrow flanges. Placement of S-I-P Deck Panels on girders with flanges less than 12 inches wide is difficult. The use of S-I-P Deck Panel shall be investigated at the preliminary design stage and proper flange width shall be considered.
5. A minimum slab thickness of 8.5 inches, including 3.5 inch precast deck panel and 5 inches C-I-P concrete topping shall be specified for design if S-I-P Deck Panels are considered for bridge deck.
6. S-I-P Deck Panels are not allowed for steel girder bridges. WSDOT's Bridge Design Engineer prefers to have a cast-in-place deck on steel girders.

5.7.4 Concrete Bridge Deck Protection Systems

A deck protection system shall be used in all projects involving concrete bridge deck construction or rehabilitation. For new bridge construction and widening projects, the type of system shall be determined by the Bridge Office Preliminary Plan and Bridge Management units during the preliminary plan stage and shall be shown on the preliminary plan in the left margin. For bridge deck rehabilitation and overlay projects, the Bridge Management Unit shall determine the type of system.

A. System Selection for New Structures

1. System 1 will be used for most New Bridges and Concrete Deck Replacements.
2. System 2 will be used on bridges with transverse post-tensioning in the deck. This system provides double protection to the post-tensioning system since restoration due to premature deck deterioration would be very costly. This system has been used on approximately 36 bridges to date that are located primarily on Interstate Routes.
3. System 3 will normally be used on bridges with precast bulb "T" girders and prestressed slabs. The ACP with membrane provides a wearing surface and some protection to the connections between the girder or slab units.

   A cast-in-place reinforced concrete overlay may be substituted for the asphalt overlay on bulb "T" or slab bridges located on routes with an ADT in excess of 20,000. The Bridge Design Engineer and the Bridge Management Unit shall be consulted to determine the best type of overlay to be used in this case.

B. Deck Overlay Selection for Bridge Widening and Existing Deck Rehabilitation

The Bridge Management Unit will recommend the type of overlay to be used on a bridge deck overlay project following discussions with the Region.

1. Epoxy-coated reinforcing will be specified in the new widened portion of the bridge.
2. The type of overlay on a deck widening shall be the same as the type used on the existing bridge. System 1 will be used on a deck widening if the existing deck does not have an overlay and no overlay is required.

3. There may be bridge widening cases when a modified concrete overlay is used on the existing bridge deck. The concrete deck profile in the widening may be constructed to match the profile of the modified concrete overlay. Contact the Bridge Preliminary Plan unit to determine if this detail applies for a bridge deck widening.
4. A modified concrete overlay will normally be used when one or both of the following criteria is met:
   a. Delaminated and patched areas of the existing concrete deck exceeding 2% of the total deck area.
   b. Exposed reinforcing steel is visible. This condition can exist on older bridges with significant traffic related wear.
5. An HMA with membrane overlay provides a short term wearing surface and low level of deck protection. This system may be used on bridges with existing HMA overlays that are to be removed and replaced.
6. Other Overlay types, such as a ¾ inch Polyester or 1 ½ inch Rapid Set LMC, are available in special cases on high ADT routes if rapid construction is needed.

C. Deck Protection Systems - New Bridges / Bridge Widenings / Bridge Deck Replacements

System 1: 2½ inches of concrete cover over epoxy-coated reinforcing.

The concrete deck is cast-in-place with no overlay. The 2½ inches of concrete cover includes a nominal depth for traction striations in the roadway surface and ¼ inch tolerance for the placement of the reinforcing steel.

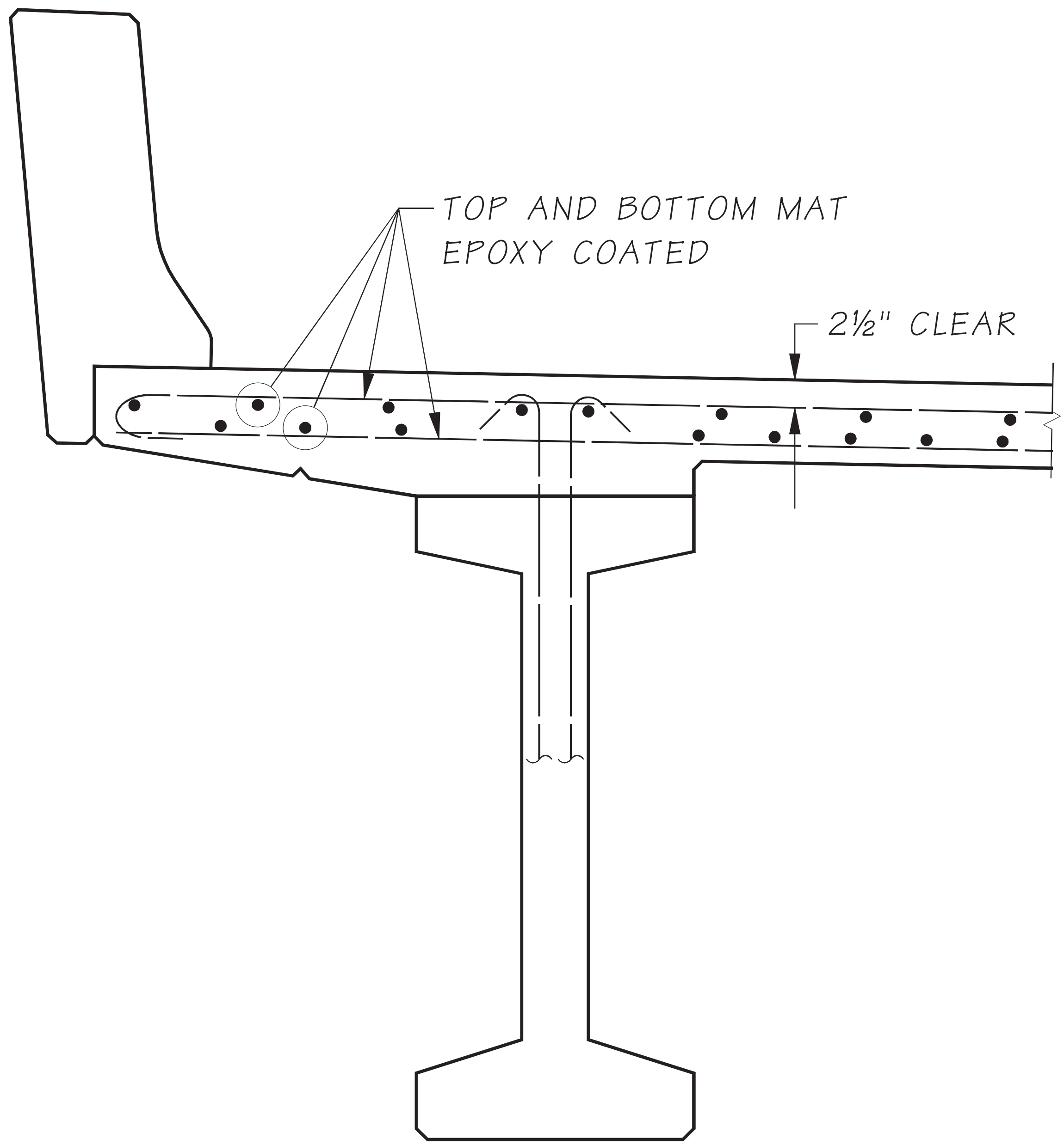


Figure 5.7.4-1

Only deck steel reinforcing mat and traffic barrier S1 bars are epoxy coated. Indicate the epoxy-coated reinforcing on the plan sheets and with an "E" in the "Epoxy Coated" column of the bar list. Add a note to the traffic barrier sheet to epoxy coat the S1 bars. See Figure 5.7.4-1.

System 2: 1¾ inches of concrete cover over epoxy-coated top mat of deck reinforcing and a 1½ inch Modified Concrete Overlay. See Figure 5.7.4-2

The concrete deck is cast-in-place. The top surface is built with 1¾ " clear, then ¼ " of the concrete deck surface is scarified prior to the placement of the 1½" Modified Concrete Overlay. The final nominal concrete cover over the top mat reinforcing is 3 inches. The type of modified concrete overlay will be specified in the contract special provisions. Generally, the contractor will be allowed to choose between Latex, Microsilica, or Fly Ash modified concrete.

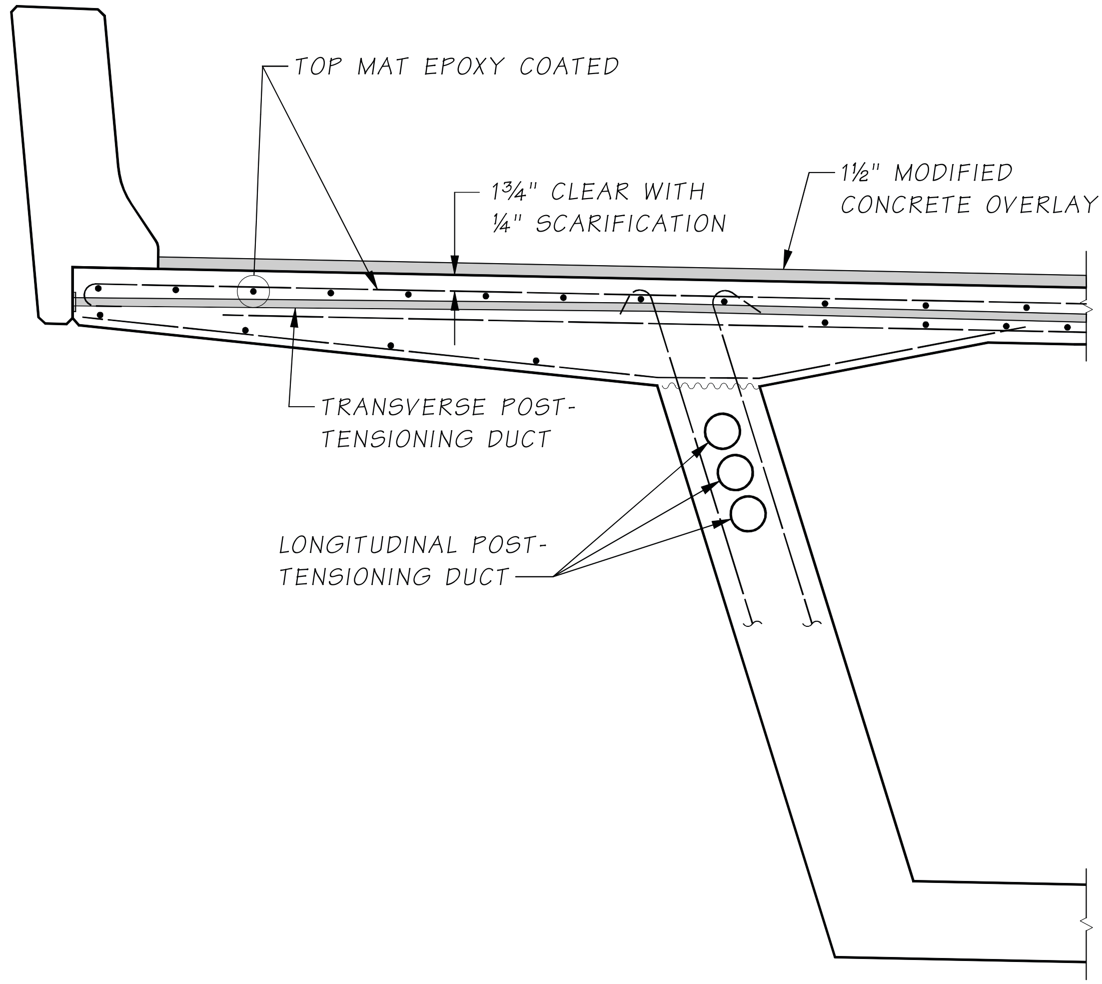


Figure 5.7.4-2

Only the bridge deck top steel reinforcing mat and traffic barrier S1 bars are epoxy coated. Indicate the epoxy-coated reinforcing on the plan sheets and with an "E" in the "Epoxy Coated" column of the bar list. Add a note to the traffic barrier sheet to epoxy coat the S1 bars.

System 3:—2-inches of concrete cover over epoxy-coated top mat of deck reinforcing with a 0.15' - 0.25' HMA with waterproofing membrane overlay. See Figure 5.7.4-3 and 5.7.4-4.

The 2-inches of concrete cover is used for precast prestressed deck members due to the use of high quality concrete and better control of reinforcing placement. The 2 inches of concrete cover includes a ¼-inch tolerance for the placement of the reinforcing steel.

The total asphalt thickness will be determined during the preliminary plan development by contacting Region Design Office. The 0.25' ACP overlay thickness is preferred if the additional deadload can be accommodated. The 0.25' of ACP will allow future overlays to remove and replace 0.15' without damaging the original membrane.

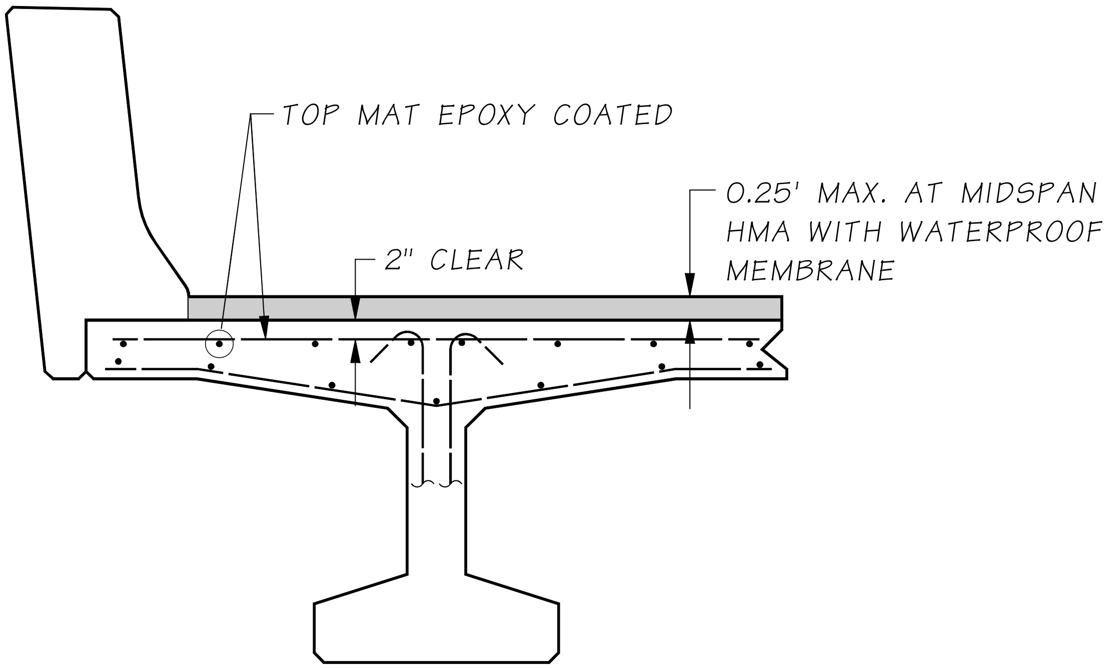


Figure 5.7.4.3

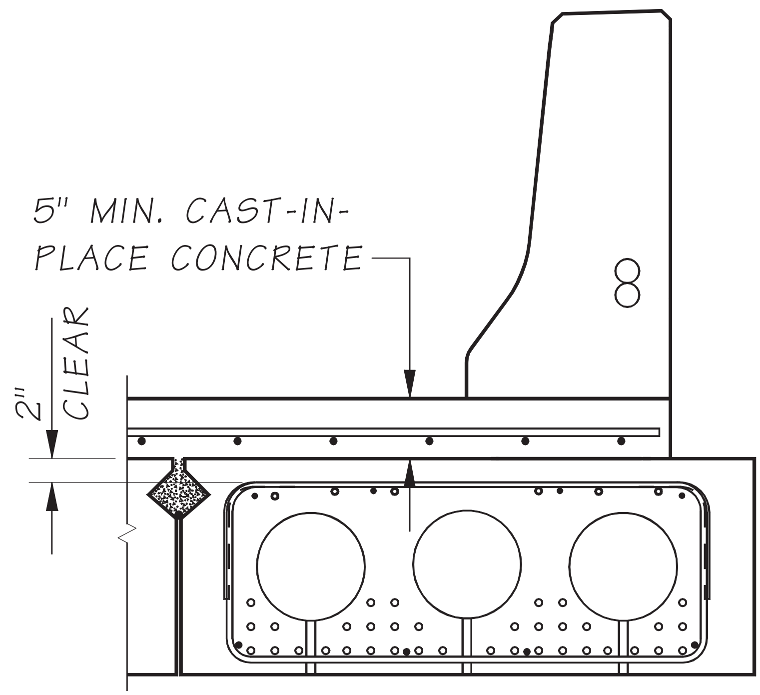


Figure 5.7.4-4

Other Systems: There may be special conditions (i.e. a widening) where it may be desirable to use a different overlay or rebar cover thickness than those shown in the typical previous Systems. For example, there have been some System 3 cases that decreased the amount of rebar cover and used a concrete overlay in order to minimize the total dead load and improve long-term performance on a high ADT route.

The Bridge Design Engineer and the Bridge Management Unit shall be consulted before one of these "Other Systems" is considered for use.

D. Deck Protection Systems – Existing Bridge Rehabilitation / Overlay

The Bridge Management unit will determine the type of overlay on deck overlay projects.

Modified Concrete Overlay – 1½ inches

A 1½ inch Modified Concrete Overlay is the preferred overlay system for providing long-term deck protection and a durable wearing surface. The Modified Concrete Overlay special provision allows a contractor to choose between a Latex, Microsilica or Fly Ash mix design. This overlay requires a deck temperature between 45 - 75 degrees and a wind speed less than 10 mph during construction. The time to construct and cure (42 hours) this overlay along with the traffic control cost can be significant. This type of overlay was first used on a WSDOT bridge in 1979 and has an expected life between 20-30 years.

The bridge deck is scarified prior to application of the modified concrete overlay. The depth of scarification varies between ¼ to ½ inch. There are three types of machines used to scarify namely; Rottomill, Hydromill or a Super shot blaster. There are advantages and disadvantages for each machine. In some cases the Bridge Management unit will request only one of these machines be used in a project.

HMA with Membrane Overlay – 0.15 to 0.25 ft

An HMAwith membrane overlay provides a low level of deck protection. This type of overlay is generally used when an overlay is needed but the deck conditions do not warrant the use of a modified concrete overlay. This type of overlay was first used on a WSDOT bridge in 1971 and has an expected life between 8-10 years depending on the ADT. The depth of overlay can vary between 0.15 ft (1.8”) and 0.25 ft (3”). The Region should be contacted to determine the depth of HMA.

Polyester Modified Concrete Overlay – ¾ inches

This type of overlay uses specialized equipment and materials. A Polyester overlay requires dry weather with temperatures above 50 degrees during construction and normally cures in 4 hours. A Polyester overlay has been used on 12 bridges (most in the early 1990’s) and has an expected life between 20-30 years depending on the ADT. Three bridges on Interstate 5 were overlaid with polyester successfully during nighttime closures in the summer of 2002 under contract 6403. A polyester concrete overlay may be used in special cases when rapid construction is needed.

Rapid Set Latex Modified Concrete Overlay – 1½ inches

A Rapid Set LMC overlay is considered experimental. This overlay uses special cement manufactured by the CTS company based in California. A Rapid Set LMC overlay is mixed in a mobil mixing truck and is applied like a regular LMC overlay. This overlay generally cures in 4 hours (verses 42 hours for a modified concrete overlay). The first Rapid Set LMC was applied to bridge 162/20 South Prairie Creek in the summer of 2002 under contract 6395. This overlay should be used on a limited basis until more is known about its long-term performance.

Thin Polymer Overlay – ½ inches

The use of Thin Polymer Overlays has generally been discontinued due to poor performance. This system has been used on movable bridges and bridges with low vertical clearances.

Appendix 5-B1: "A" Dimension for Precast Girder Bridges

Introduction

The girder haunch is the distance between the top of a girder and the bottom of the roadway slab. The haunch varies in depth along the length of the girder accommodating the girder camber and geometric effects of the roadway surface including super elevations, vertical curves and horizontal curves.

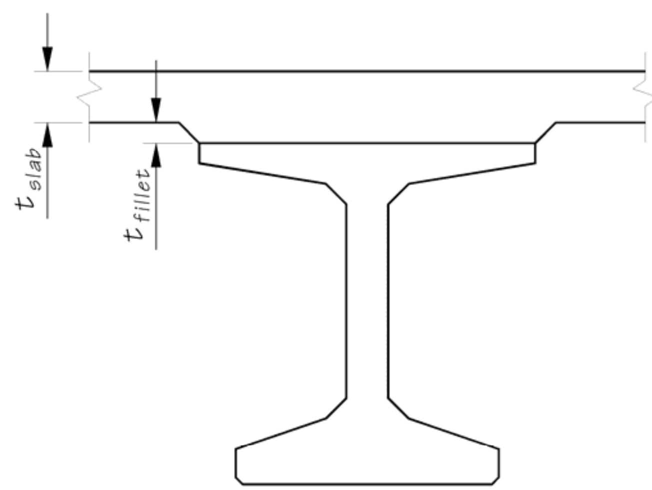
The basic concept in determining the required "A" dimension is to provide a haunch over the girder such that the top of the girder is less than the fillet depth (typically ¾") below the bottom of the slab at the center of the span. This provides that the actual girder camber could exceed the calculated value by 1¾" before the top of the girder would interfere with the bottom mat of slab reinforcement.

It is desirable to have points of horizontal and vertical curvature and super elevation transitions off the bridge structure as this greatly simplifies the geometric requirements on the girder haunch. However, as new bridges are squeezed into the existing infrastructure it is becoming more common to have geometric transitions on the bridge structure.

Each geometric effect is considered independently of the others. The total geometric effect is the algebraic sum of each individual effect.

Fillet Effect

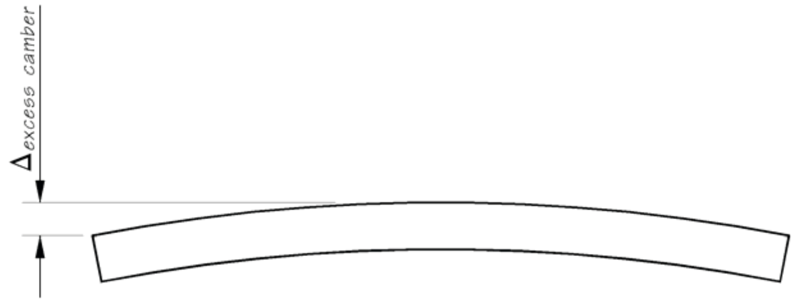
The distance between the top of the girder and the top of the roadway surface, must be at least the thickness of the roadway slab plus the fillet depth.



$$\Delta_{deck} = t_{slab} + t_{fillet}$$

Excessive Camber Effect

The girder haunch must be thickened to accommodate any camber that remains in the girder after slab casting. This is the difference between the "D" and "C" dimensions from the Girder Schedule Table. Use a value of 2 ½" at the preliminary design stage to determine vertical clearance.



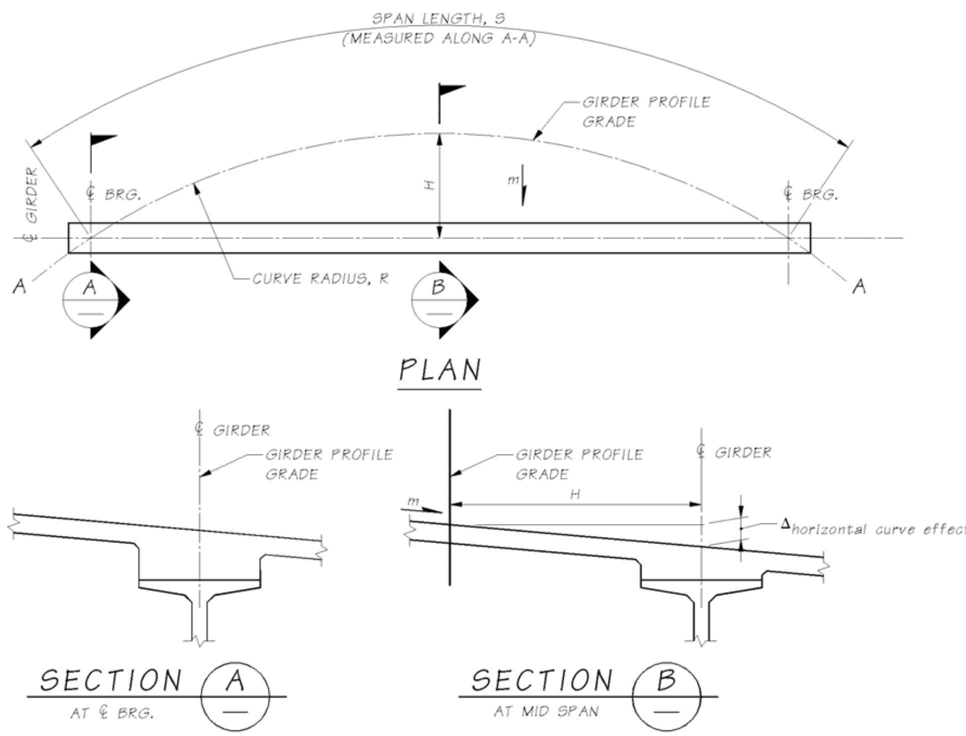
Profile Effect

The profile effect accounts for changes in the roadway profile along the length of the girder. Profile changes include grade changes, vertical curve effects, and offset deviations between the centerline of girder and the alignment caused by flared girders and/or curvature in the alignment.

When all of the girders in a span are parallel and the span is contained entirely within the limits of a vertical and/or horizontal curve, the profile effect is simply the sum of the Vertical Curve Effect and the Horizontal Curve Effect.

$$\Delta_{profile\ effect} = \Delta_{vertical\ curve\ effect} + \Delta_{horizontal\ curve\ effect}$$

The horizontal curve effect is, assuming a constant super elevation rate along the length of the span, $\Delta_{horizontal\ curve\ effect} = \frac{1.5S^2m}{R}$ where S is the length of curve in feet, R is the radius of the curve in feet, and m is the crown slope. The horizontal curve effect is in inches.



$$\Delta = \frac{S}{R}$$

$$\phi = \frac{\Delta}{4}$$

$$\phi = \frac{S}{4R}$$

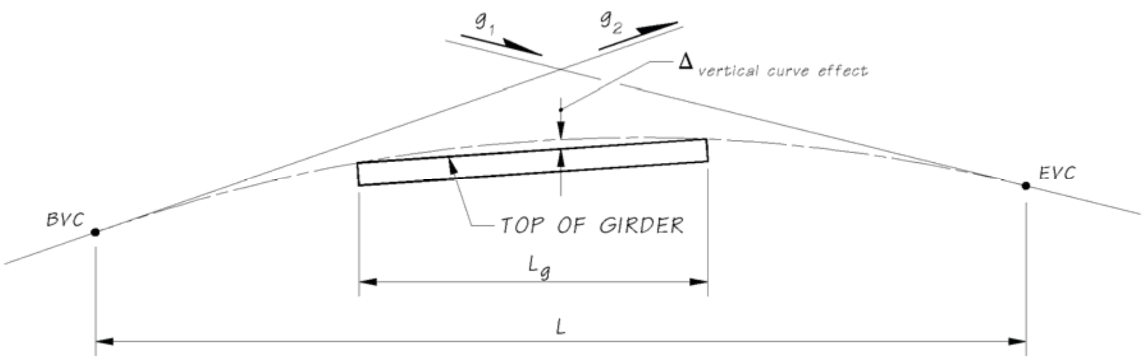
$$\tan\phi \approx \phi$$

$$\tan\phi \approx \frac{2H}{S}$$

$$H = \frac{S}{2}\tan\phi \approx \frac{S}{2}\phi = \frac{S}{2}\times\frac{S}{4R} = \frac{S^2}{8R}$$

$$\Delta_{horizontal\ curve\ effect} = \frac{S^2}{8R} m \times 12\,\tfrac{in}{ft} = \frac{1.5S^2}{R} m\left(inches\right)$$

The vertical curve effect is $\Delta_{vertical\ curve\ effect} = \dfrac{1.5GL_g^2}{100L}$ where G is the algebraic difference in profile tangent grades ($G = g_2 - g_1$) (%), Lg is the girder length (feet), and L is the vertical curve length (feet). The vertical curve effect is in inches and is positive for sag curves and negative for crown curves.



$$K = \frac{100G}{2L}$$

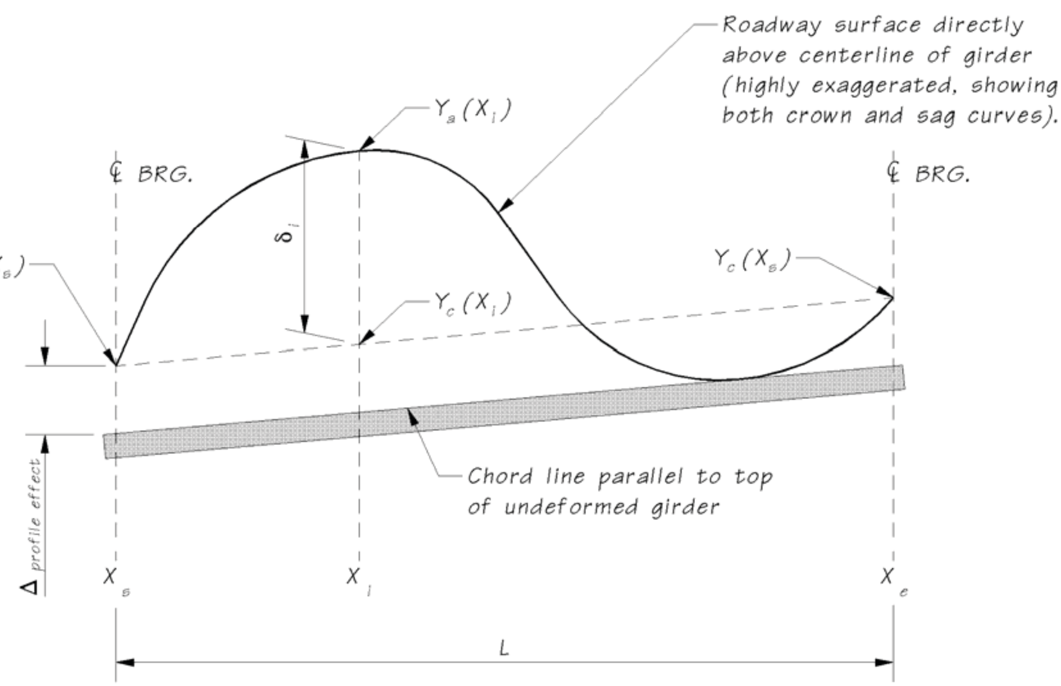
$$\Delta_{vertical\ curve\ effect} = K\frac{L_g^2}{40{,}000}\times 12\,\tfrac{in}{ft} = \frac{G}{2L}\times\frac{L_g^2}{400}\times 12 = \frac{1.5GL_g^2}{100L}$$

If one or more of the following roadway geometry transitions occur along the span, then a more detailed method of computation is required:

- change in the super elevation rate
- grade break
- point of horizontal curvature
- point of vertical curvature
- flared girders

The exact value of the profile effect may be determined by solving a complex optimization problem. However it is much easier and sufficiently accurate to use a numerical approach.

The figure below, while highly exaggerated, illustrates that the profile effect is the distance the girder must be placed below the profile grade so that the girder, ignoring all other geometric effects, just touches the lowest profile point between the bearings.



In the case of a crown curve the haunch depth may reduced. In the case of a sag curve the haunch must be thickened at the ends of the girder.

To compute the profile effect:

1. Create a chord line parallel to the top of the girder (ignoring camber) connecting the centerlines of bearing. The equation of this line is

$$y_c(x_i) = y_a(x_s, z_s) + (x_i - x_s)\left(\frac{y_a(x_e, z_e) - y_a(x_s, z_s)}{x_e - x_s}\right)$$

where

x_i	=	Station where the elevation of the chord line is being computed
x_s	=	Station at the start of the girder
x_e	=	Station at the end of the girder
z_s	=	Normal offset from alignment to centerline of the girder at the start of the girder at station x_s
z_e	=	Normal offset from the alignment to the centerline of the girder at the end of the girder at station x_e
$y_a(x_s,z_s)$	=	Elevation of the roadway profile at station x_s and offset z_s
$y_a(x_e,z_e)$	=	Elevation of the roadway profile at station x_e and offset z_e
$y_c(x_i)$	=	Elevation of the chord line at station x_i

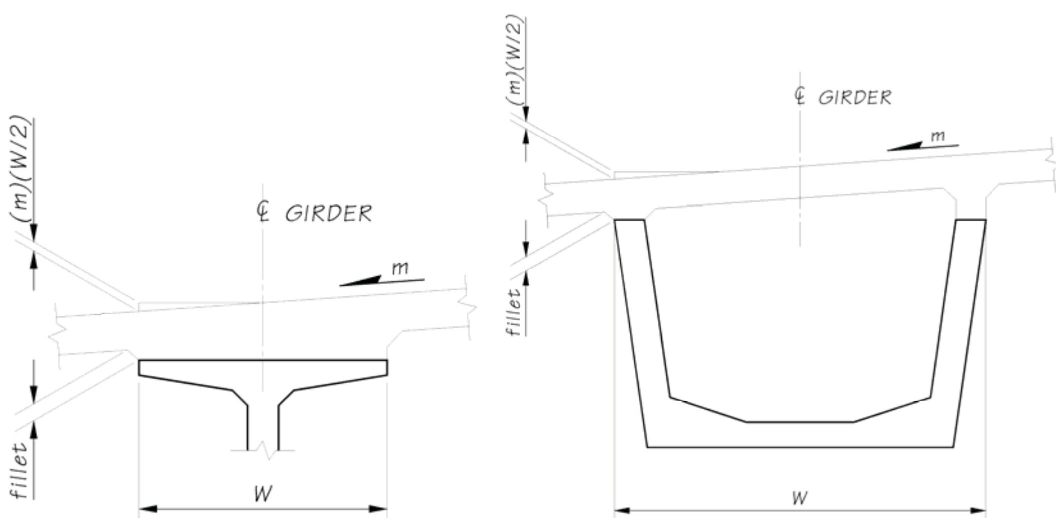
2. At 10th points along the span, compute the elevation of the roadway surface directly above the centerline of the girder, $y_a(x_i,z_i)$, and the elevation of the line paralleling the top of the girder, $y_c(x_i)$. The difference in elevation is the profile effect at station x_i, $\Delta_{profile\,effect\,@\,i} = y_a(x_i, z_i) - y_c(x_i)$.

Girder Orientation Effect

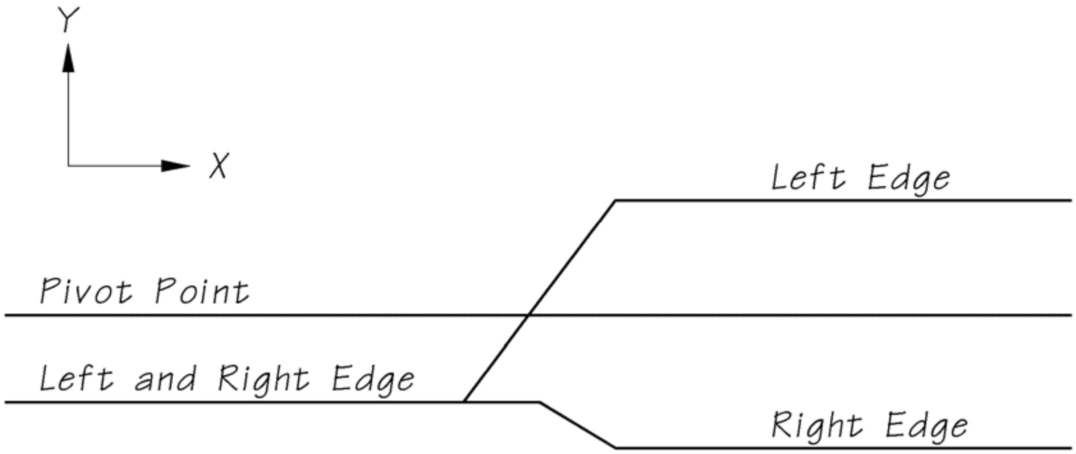
The girder orientation effect accounts for the difference in slope between the roadway surface and the top of the girder. Girders such as I-beams are oriented with their Y axis plumb. Other girders such as U-beam, box beam, and slabs are oriented with their Y axis normal to the roadway surface. The orientation of the girder with respect to the roadway surface, and changes in the roadway surface along the length of the girder (super elevation transitions) define the Girder Orientation Effect.

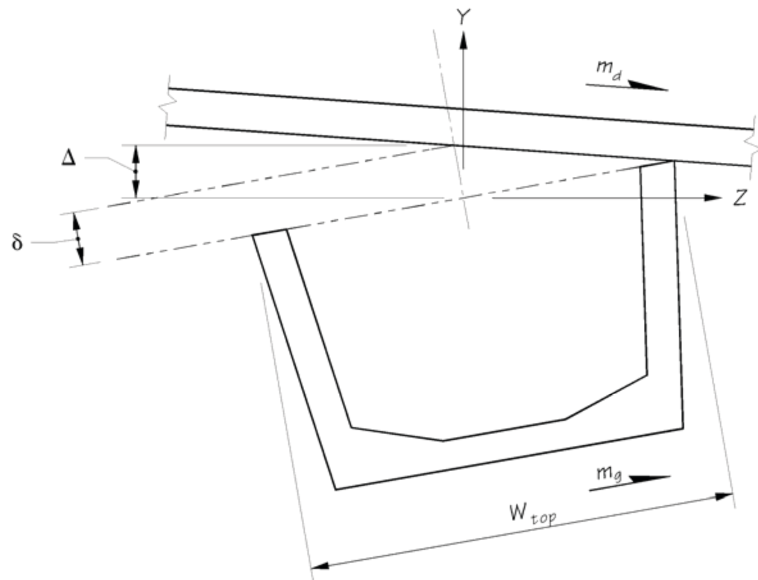
If the super elevation rate is constant over the entire length of the span and the Y-axis of the girder is plumb, the girder orientation effect simplifies to the Top Width Effect,

$$\Delta_{girder\ orientation\ effect} = \Delta_{top\ width\ effect} = m\left(\frac{W_{top}}{2}\right).$$



If there is a change in super elevation rate and/or the Y-axis of the girder is not plumb, then once again a more complex computation is required.





To compute the girder orientation effect at each 10th point along the girder:

1. Determine the cross slope, m, of the roadway surface at station x_i. If there is a crown point over the girder the cross slope is taken as

$$m(x_i, z_i) = \frac{y_a(x_i, z_i^{left}) - y_a(x_i, z_i^{right})}{z_i^{left} - z_i^{right}}$$

where

x_i	=	The station where the cross slope is being computed
z_i	=	Normal offset from the alignment to the centerline of the girder at the end of the girder at station x_i
z_i^{left}	=	Offset from the alignment to the top left edge of the girder
z_i^{right}	=	Offset from the alignment to the top right edge of the girder
$y_a(x_i, z_i^{left})$	=	Roadway surface elevation at station x_i and normal offset z_i^{left}
$y_a(x_i, z_i^{right})$	=	Roadway surface elevation at station x_i and normal offset z_i^{right}

2. Determine the angle between the Y-axis of the girder and the roadway surface at station x_i. $\theta_{x_i} = \tan^{-1}\left(\left|m(x_i, z_i) - m_g\right|\right)$, where m_g is the slope of the girder.

3. Determine the girder orientation effect at station x_i. $\Delta_{girder\,orientation\,effect\,@i} = \dfrac{(W_{top})(\tan\theta_{x_i})}{2\cos\theta_{x_i}}$

"A" Dimension

The "A" dimension is the sum of all these effects.

$A = \Delta_{fillet} + \Delta_{excess\,camber} + \Delta_{profile\,effect} + \Delta_{girder\,orientation\,effect}$

If you have a complex alignment, determine the required "A" dimension for each section and use the greatest value.

Round "A" to the nearest ¼".

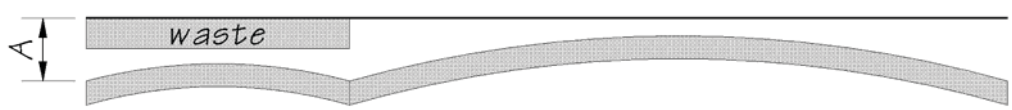
The minimum value of "A" is $A_{\min} = \Delta_{fillet} + \Delta_{girder\,orientation\,effect}$.

If a Drain Type 5 crosses the girder, "A" shall not be less than 9 inches.

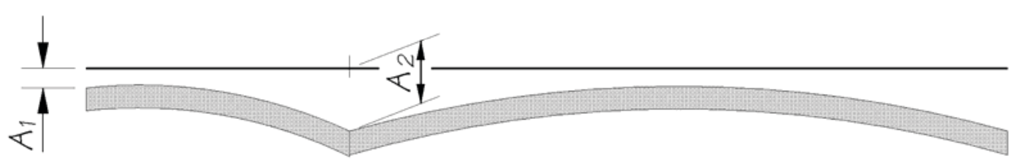
Limitations

These computations are for a single girder line. The required haunch should be determined for each girder line in the structure. Use the greatest "A" dimension.

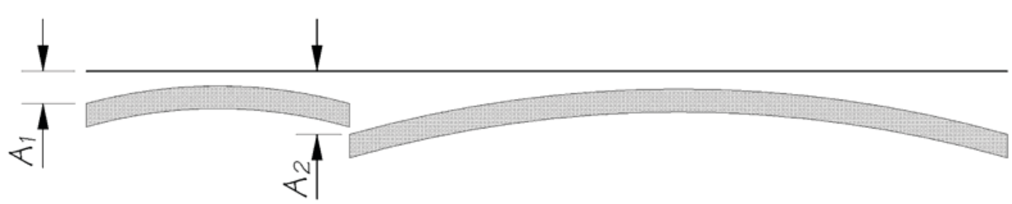
These computations are also limited to a single span. A different haunch may be needed for each span or each pier. For example, if there is a long span adjacent to a short span, the long span may have considerably more camber and will require a larger haunch. There is no need to have the shorter spans carry all the extra concrete needed to match the longer span haunch requirements. With the WF series girders, the volume of concrete in the haunches can add up quickly. The shorter span could have a different haunch at each end as illustrated below.



Same "A" dimension at all piers



Pier by pier "A" dimensions



Span by span "A" dimensions

Stirrup Length and Precast Deck Leveling Bolt Considerations

For bridges on crown vertical curves, the haunch depth can become excessive to the point where the girder and diaphragm stirrups are too short to bend into the proper position. Similarly the length of leveling bolts in precast deck panels may need adjustment.

Stirrup lengths are described as a function of “A” on the standard girder sheets. For example, the G1 and G2 bars of a WF74G girder are 6’-5”+ “A” in length. For this reason, the stirrups are always long enough at the ends of the girders. Problems occur when the haunch depth increases along the length of the girder to accommodate crown vertical curves and super elevation transitions.

If the haunch depth along the girder exceeds “A” by more than 2 inches, an adjustment must be made. The haunch depth at any section can be compute as $A - \Delta_{profile\ effect} - \Delta_{excess\ camber}$.

“A” Dimension Worksheet - Simple Alignment

Fillet Effect

Slab Thickness (t_{slab}) = ______ in
Fillet Size (t_{fillet}) = ______ in
$\Delta_{fillet} = t_{slab} + t_{fillet}$ = ______ in

Excess Camber Effect

“D” Dimension from Girder Schedule (120 days) = ______ in

“C” Dimension from Girder Schedule = ______ in

$\Delta_{excess\ camber}$ = “D” - “C” = ______ in

Profile Effect

Horizontal Curve Effect, $\Delta_{horizontal\ curve\ effect} = \dfrac{1.5S^2m}{R}$ = ______ in

Vertical Curve Effect, $\Delta_{vertical\ curve\ effect} = \dfrac{1.5GL_g^2}{100L}$ = ______ in
(+ for sag, – for crown)

$\Delta profile = \Delta horizontal\ curve\ effect + \Delta vertical\ curve\ effect$ = ______ in

Girder Orientation Effect

Girder must be plumb.

$\Delta girder\ orientation$ = 0 for U-beams inclined parallel to the slab.

$\Delta girder\ orientation = \Delta top\ flange\ effect = m\left(\dfrac{W_{top}}{2}\right)$ = ______ in

“A” Dimension

$\Delta_{fillet} + \Delta_{excess\ camber} + \Delta_{profile\ effect} + \Delta_{girder\ orientation\ effect}$ = ______ in

Round to nearest ¼” = ______ in

Minimum “A” Dimension, $\Delta_{fillet} + \Delta_{girder\ orientation\ effect}$ = ______ in

“A” Dimension = ______ in

Example

Slab: Thickness = 7.5", Fillet = 0.75"

WF74G Girder: W_{top} = 49"

Span Length = 144.4 ft

Crown Slope = 0.04ft/ft

Camber: D = 7.55", C = 2.57"

Horizontal Curve Radius = 9500ft through centerline of bridge

Vertical Curve Data: g_1 = 2.4%, g_2 = -3.2%, L = 800ft

Fillet Effect

Slab Thickness (t_{slab}) = 7.5 in

Fillet Size (t_{fillet}) = 0.75 in

$\Delta fillet = t_{slab} + t_{fillet}$ = 8.25 in

Excess Camber Effect

"D" Dimension from Girder Schedule (120 days) = 7.55 in

"C" Dimension from Girder Schedule = 2.57 in

$\Delta_{excess\ camber}$ = "D" - "C" = 4.98 in

Profile Effect

Horizontal Curve Effect,

Chord Length = 144.4ft, $C = 2R\sin\frac{\Delta}{2}$ $144.4 = 2(9500)\sin\frac{\Delta}{2}$ $\Delta = 0.87"$

Curve Length = $R\Delta\frac{\pi}{180} = 9500(0.87)\frac{\pi}{180} = 144.4\,ft$

$$\Delta_{horizontal\ curve\ effect} = \frac{1.5S^2m}{R} = \frac{1.5(144.4)^2\,0.04}{9500} = 0.13in$$

Vertical Curve Effect,

$$\Delta_{vertical\ curve\ effect} = \frac{1.5GL_g^2}{100L} = \frac{1.5(-5.6)(144.4)^2}{100(800)} = -2.19in$$

(+ for sag, – for crown)

$\Delta_{profile} = \Delta_{horizontal\ curve\ effect} + \Delta_{vertical\ curve\ effect} =$ 0.13-2.19 = -2.06in

Girder Orientation Effect

$$\Delta_{girder\ orientation} = \Delta_{top\ flange\ effect} = m\left(\frac{W_{top}}{2}\right) = 0.04\frac{49}{2} = 0.98in$$

"A" Dimension

$\Delta_{fillet} + \Delta_{excess\ camber} + \Delta_{profile\ effect} + \Delta_{girder\ orientation\ effect} =$ 8.25 + 4.98 – 2.06 + 0.98 = 12.15 in

Round to nearest ¼" = 12.25 in

Minimum "A" Dimension, $\Delta_{fillet} + \Delta_{girder\ orientation\ effect} =$ 8.25 + 0.98 = 9.23 in

"A" Dimension = 12¼ in

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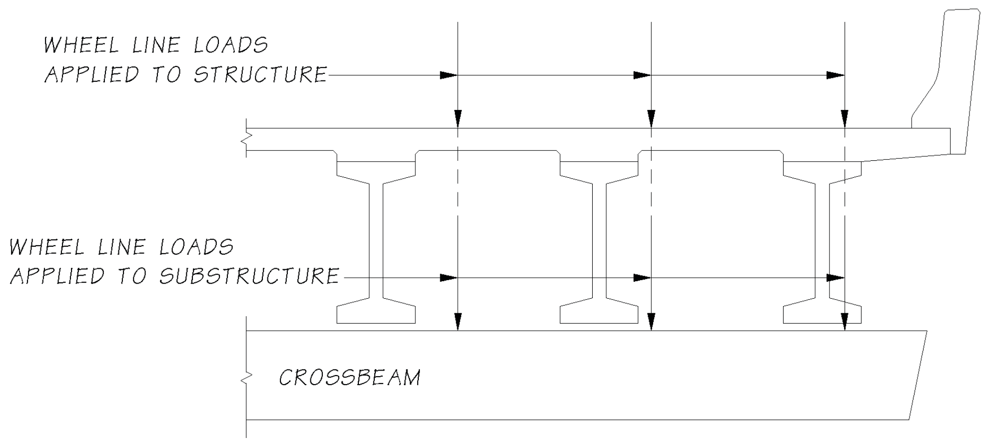
Chapter 7 Substructure Design

B. Live Loads - LL

The dynamic allowance (IM) will be applied in accordance with AASHTO 3.6.2 and is not included in the design of buried elements of the substructure. Portions of the abutments in contact with the soil are considered buried elements.

Lane reduction factors as described in AASHTO “Reduction in Load Intensity” are applied to the number of lanes for each load case.

The HL93 loading is distributed to the substructure by placing wheel line reactions in a lane configuration that generates the maximum stress in the substructure. A wheel line reaction is ½ of the HL93 reaction. Live loads are considered to act directly on the substructure without further distribution through the superstructure, see Figure 7.1.3-2. Normally, substructure design will not consider live load torsional or lateral distribution nor any live load sidesway effects. GTSTRUDL will include live load sidesway.



Live Load Wheel Line Destribution to Substructure

Figure 7.1.3-2

For steel and prestressed concrete superstructures where the live load is transferred to substructure through bearings, cross frames or diaphragms, the girder reaction may be used for substructure design.

Live load placement is dependant on the member under design. Some examples of live load placement are as follows.

The exterior vehicle wheel is placed 2 feet from the curb for maximum crossbeam cantilever moment or maximum eccentric foundation moment.

For crossbeam design between supports, the HL93 lanes are placed to obtain the maximum moment in the member; then re-located to obtain the maximum shear or negative moment in the member.

For column design, the design lanes are placed to obtain the maximum transverse moment at the top of the column; then re-located to obtain the maximum axial force of the column.

C. Down Drag Force - DD

The Geotechnical Report will provide the downdrag force. The down drag force (DD) is a load applied to the pile/shaft with the load factor (γ_{DD}) specified in the Geotechnical Report. Generally, transient loads (LL) are less than the downdrag force and should be omitted when considering DD forces. In other words, the transient loads reduce downdrag forces and are ignored for the structural design. The WSDOT GDM Section 8.6.2 provides a more in-depth discussion of Down Drag.

D. Earthquake Loads - EQ

Earthquake loads on elements of the substructure are described in AASHTO 3.10, Earthquake Effects. The design acceleration coefficient and site coefficient will be given in the Geotechnical Report. The seismic analysis requirements are stated in AASHTO Section 4.7. The resulting loads shall be taken in any horizontal direction to give maximum design load for the substructure element.

The intermediate pier(s) of each unit of a multispan continuous structure shall be designed to resist the entire longitudinal earthquake force for that unit (unless the end piers are an integral part of the superstructure). The calculated longitudinal movement shall be used to determine the shear force developed by the pads at the abutments. The neoprene modulus of elasticity at 70°F (21°C) shall be used by determine the shear force. However, the force transmitted through a bearing pad shall be limited to the force that causes the pad to slip.

Hold-Down Devices shall be designed per AASHTO 3.10.9.6. This requires a minimum earthquake force to cause uplift on the substructure equal to 10 percent of the dead load reaction of the superstructure. Where such forces can be developed, the crossbeam, column and footing shall be designed to carry these earthquake forces.

For concrete superstructures built integrally with the substructure, the substructure elements shall be designed to carry their dead load plus all the elements below them including soil overburden as though they were suspended from the superstructure. (Seal not included). For this condition, the ultimate downward force shall be 1.0 (EQ + Uplift). For structures carried on elastomeric pads or where there is no positive vertical connection, the uplift force from the superstructure shall be neglected.

E. Post-tensioning Effects from Superstructure - EL

When cast-in-place, post-tensioned superstructures are constructed monolithic with the piers, the substructure design should take into account frame moments and shears caused by elastic shortening and creep of the superstructure upon application of the axial post-tensioning force at the bridge ends. Frame moments and shears thus obtained should be added algebraically to the values obtained from the primary and secondary moment diagrams applied to the superstructure.

When cast-in-place, post-tensioned superstructures are supported on sliding bearings at some of the piers, the design of those piers should include the longitudinal force from friction on the bearings generated as the superstructure shortens during jacking. When post-tensioning is complete, the full permanent reaction from this effect should be included in the governing AASHTO load combinations for the pier under design.

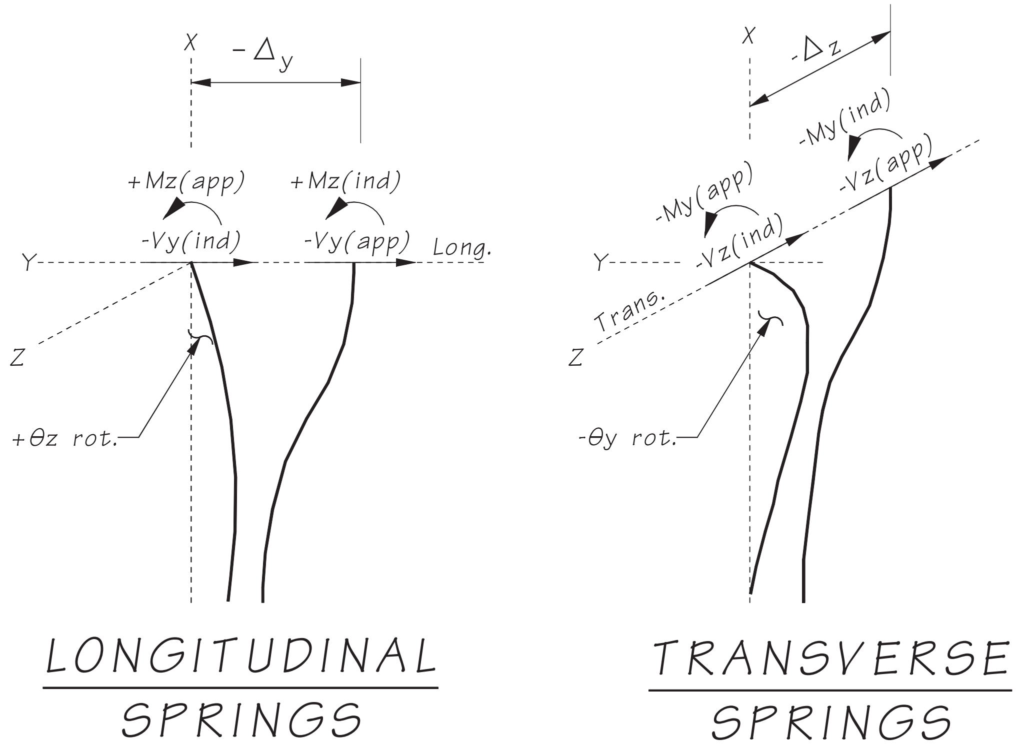


Figure 7.2.5-9

Where the linear spring constants or K values are defined as:

K11	= -Vy/-Δy	= positive = Kip/in	= Longitudinal Lateral Stiffness
K22	= AE/L	= positive = Kip/in	= Vertical or Axial Stiffness
K33	= -Vz/-Δz	= positive = Kip/in	= Transverse Lateral Stiffness
K44	= -My/-θy	= positive = K in/rad	= Transverse Bending or Moment Stiffness
K55	= JG/L	= positive = K in/rad	= Torsional Stiffness
K66	= Mz/θz	= positive = K in/rad	= Longitudinal Bending or Moment Stiffness
K34	= $-Vz_{ind}$/-θ y	= positive = Kip /rad	= Trans. shear X-couple term (Fixed Head only)
K16	= $-Vy_{ind}$ /θz	= negative = Kip /rad	= Long. shear X-couple term (Fixed Head only)
K43	= -Myind /-Δz	= positive = Kip /rad	= Long. Moment X-couple term (Fixed Head only)
K61	= $+Mz_{ind}$ /-Δy	= negative = Kip /rad	= Trans. Moment X-couple term (Fixed Head only)

1. Lateral Lpile Springs: K11 and K33

   The loading should apply the lateral load (shear = V) and restrain the top against rotation (slope = 0).

Boundary condition code	=	2
Lateral load at the pile head	=	0.250D+05 lbs
Slope at the pile head	=	0.000D+00 in/in
Axial load at the pile head	=	0.758D+05 lbs = P

X	Deflection	Moment	Shear	Soil Reaction	Total Stress	Flexural Rigidity
In	In	Lbs-In	Lbs	Lbs-In	Lbs/In**2	Lbs-In**2
******	**********	**********	**********	**********	**********	**********
0.00	0.267D+01	-0.383D+07	0.250D+05	0.000D+00	0.270D+05	0.392D+11

$-Vy_{app}$ or $-Vz_{app}$ = 25 kip

$-\Delta y$ or $-\Delta z$ = 2.67"

K11 = $-Vy/-\Delta y$ and is equal to K33 = $-Vz/-\Delta z$=

Note the induced moment values for cross-couple calculation are $+Mz_{ind}$ or $-My_{ind}$ = 3830 k-in.

2. Moment Lpile Springs: K44 and K66

The loading should apply a moment (M) and restrain the pile top against translation (deflection = 0).

Boundary condition code	=	4
Deflection at the pile head	=	0.000D+00 in
Moment at the pile head	=	0.391D+07 in-lbs
Axial load at the pile head	=	0.103D+06 lbs

X	Deflection	Moment	Shear	Soil Reaction	Total Stress	Flexural Rigidity
In	In	Lbs-In	Lbs	Lbs-In	Lbs/In**2	Lbs-In**2
**********	**********	**********	**********	**********	**********	**********
0.00	0.000D+00	0.391D+07	0.189D+05	0.000D+00	0.281D+05	0.392D+11

$+Mz_{app}$ or My_{app} = 391 K-in

$+\theta z$ or $-\theta y$ = 0.00845 rad

K44 = $+Mz/+\theta z$ and is equal to K66 = $-My/-\theta y$=

Note the induced shear values for cross-couple calculation are $-Vy_{ind}$ or $-Vz_{ind}$=18.9 kip

3. Cross Couple Springs: K34 and K16

Given $-Vy_{ind}$ or $-Vz_{ind}$ = 18.9 kip and $+\theta z$ or $-\theta y$ = 0.00845 rad

Given $+Mz_{ind}$ or $-My_{ind}$= 3830 k-in and $-\Delta y$ or $-\Delta z$ = 2.67"

Transverse K34 = $-Vz_{ind}/-\theta y = \dfrac{-18.9kip}{-0.00845rad} = 2236\,\dfrac{kip}{rad}$

Transverse K43 = $-My_{ind}/-\Delta z = \dfrac{-3830\ kip\ in}{-2.67\ in} = 1434\,\dfrac{kip}{rad}$

Use average value for cross-couple: K34 = K43 $= \dfrac{2236 + 1434}{2} = 1835\,\dfrac{kip}{rad}$

Longitudinal K16 = $-Vy_{ind}/+\theta z = \dfrac{-18.9kip}{0.00845rad} = -2236\,\dfrac{kip}{rad}$

Longitudinal K61 = $+Mz_{ind}/-\Delta y = \dfrac{3830\ kip\ in}{-2.67\ in} = -1434\,\dfrac{kip}{rad}$

Use average value for cross-couple: K16 = K61 $= \dfrac{-2236 + -1434}{2} = -1835\,\dfrac{kip}{rad}$

Contents

Chapter 13 Bridge Load Rating

E. Impact (LRFR)

For new bridge designs, impact shall be 10 percent (0.1).

For existing bridges, the impact factor shall be determined by the approach roadway and the deck condition. For approach roadway condition codes 6 or greater, assume 10 percent impact; for codes less than 6, assume 20 percent impact. If the bridge deck condition is 6 or greater or has 0 to 4 percent scaling, assume 10 percent impact; if the deck condition is 5 or has between 5 and 15 percent scaling, assume 20 percent impact; if the deck condition is 4 or less and has greater

F. Live Load Reduction Factors (LRFR)

Number of Loaded Lanes	Reduction Factor
One or two lanes	1.0
Three lanes	0.8
Four lanes or more	0.7

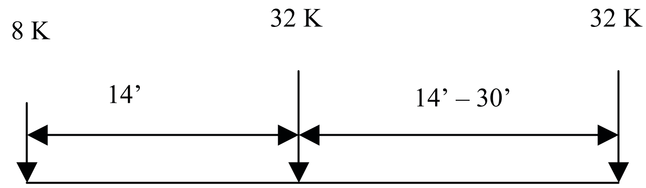
G. Live Loads (LRFR)

The moving loads for the rating shall be the HS-20 truck/lane loading (Figure 13.1.1.8-1), three legal trucks/ lane load (Figure 13.1.1.8-2), and two overload trucks. (Figure 13.1.1.8-3). The legal lane load shall be used to rate structures with spans <u>over</u> 200 feet. For the two overload trucks (OL-1 and OL-2), use only one overload truck occupying one lane in combination with one of the AASHTO legal trucks in each of the remaining lanes, when modeling the full section of the bridge or cross-beams. The number of lanes used shall be the actual striped lanes at the time of rating.

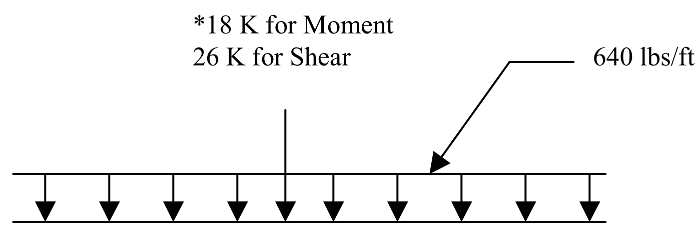
The three legal trucks and legal lane load, Type 3, Type 3S2, and Type 3–3, are to be used to determine posting limits. The two overload vehicles represent extremes in the limits of permitted vehicles in Washington State.

H. Rating Trucks

Design Trucks



HS-20 Truck

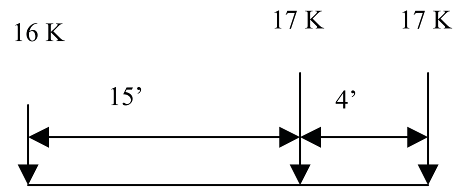


HS-20 Lane Load

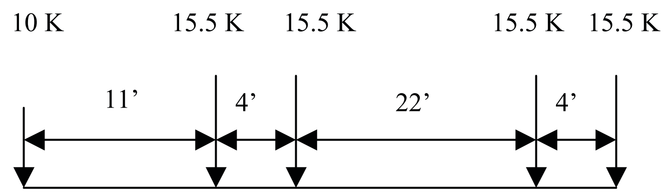
* In negative moment regions of continuous spans, place an equivalent load in the other span to produce the maximum effect.

Figure 13.1-1

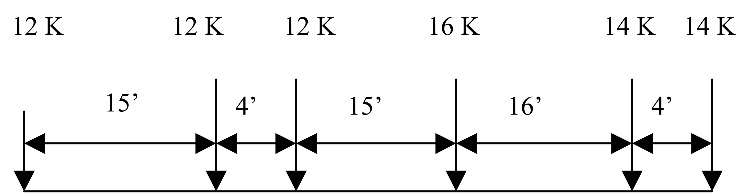
Legal Trucks



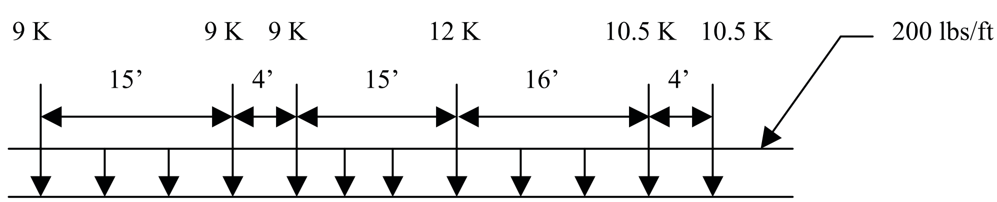
Type 3 Truck



Type 3S2 Truck



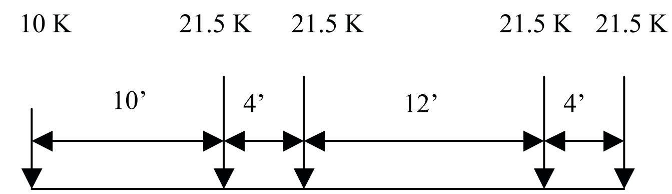
Type 3-3 Truck



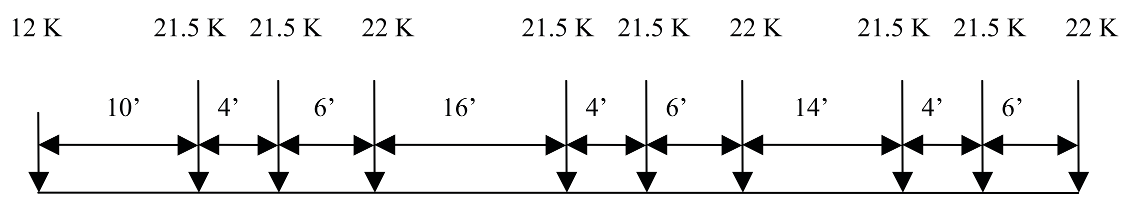
Legal Lane Load

Figure 13.1-2

Overload Trucks



Overload 1



Overload 2

Figure 13.1-3

13.1.2 NBI Rating (LFR)

Ratings shall be performed per the latest AASHTO *Manual for Condition Evaluation of Bridges*. All bridges, except timber, shall be rated based on the Load Factor method. The HS20 Truck/ Lane shall be used to calculate the Inventory and Operating Ratings.

A. Strength Method (LFR)

The basic equation shall be:

$$R.F. = \frac{\Phi R_n - \gamma_{DL} D \pm S}{\gamma_L L_{(1+I)}}$$

Where:

R.F. = Rating Factor (Ratio of Capacity to Demand)

R_n = Nominal Capacity of the Member

Φ = Resistance Factor (Per AASHTO Standard Specs.)

D = Unfactored Dead Load Moment or Shear

L = Unfactored Live Load Moment or Shear

S = Unfactored Prestress Secondary Moment or Shear

I = Impact Factor, Span dependant (Per AASHTO Standard Specs.)

γ_{DL} = 1.3 (Dead Load Factor)

γ_L = 2.17 for Inventory (Live Load Factor)

= 1.30 for Operating

Truck/Lane shall be used to calculate the Inventory and Operating Ratings.

B. Service Method (LFR)

1. Prestressed and Post-tensioned Members

Prestressed and post-tensioned members in positive moment regions, and where post-tensioning is continuous over the supports, shall also be rated based on allowable stresses at service loads. The lowest rating factor between Service and Load Factor methods shall be the governing Inventory rating. The Operating rating shall be based on the load factor method using a Live Load factor of 1.30. Service ratings for the HS20 shall be the same as stated in Section 13.1.1.B, except the impact factor shall be span dependant.

2. Timber Members

$$R.F. = \frac{F_A - F_D}{F_L}$$

R.F. = Rating Factor (Ratio of Capacity to Demand)

F_A = Allowable bending stress

F_D = Dead Load Stress

F_L = Stress due to Live Load, does not include Impact

* F_A, for Inventory rating shall be per AASHTO Standard Specifications. For Operating

Ratings, F_A shall be per AASHTO Standard Specifications with a 33% increase in the allowable stress.

C. Resistance Factors (LFR)

The resistance factors for NBI ratings shall be per the latest AASHTO Standard Specifications. Following are the NBI resistance factors:

Steel Members:	1.00 (Flexure) 1.00 (Shear)
Prestressed Concrete	1.00 (Flexure, Positive moment) 0.90 (Shear)
Post-tensioned, Cast in place:	0.95 (Flexure, Positive moment) 0.90 (Shear)
Reinforced Concrete:	0.90 (Flexure) 0.85 (Shear)

For prestressed and post-tensioned members, where reinforcing steel is used to resist negative moment, the resistance factors for reinforced concrete section shall be used in the ratings.

D. Live Loads

The HS-20 truck or lane shall be used to load rate bridge members. The number of lanes shall be per AASHTO Standard Specifications, Section 3.6. When multiple lanes are considered, apply the appropriate multilane reduction factor given in Section 13.1.2.F. Load distribution methods are discussed under specific bridge types. Do not consider sidewalk live loads in rating analysis.

E. Impact (LFR)

Impact is expressed as a fraction of the live load stress, and shall be determined by the following formula:

$$I = \frac{50}{125 + L}$$

I = Rating Factor (Ratio of Capacity to Demand)

L = Length in feet of the portion of the span that is loaded to produce the maximum stress in the member.

*AASHTO Standard Specifications for Highway Bridges 3.8.2.1.

F. Live Load Reduction Factors (LFR)

Number of Loaded Lanes	Reduction Factor
One or two lanes	1.0
Three lanes	0.9
Four lanes or more	0.75

13.2 Special Rating Criteria

13.2.1 Dead Loads

Dead Loads shall be as defined in the AASHTO *Standard Specifications for Highway Bridges*, except concrete weight shall be 155 pcf.

13.2.2 Live Load Distribution Factors

Live Load distribution factors shall be per Chapter 3 of the AASHTO *Standard Specifications for Highway Bridges.* Distribution factors are selected assuming one traffic lane where the roadway is less than 20 feet wide or two or more traffic lanes where the roadway is 20 feet or wider.

13.2.3 Reinforced Concrete Structures

For conventional reinforced concrete members of existing bridges, checking of serviceability shall not be part of the rating evaluation.

Rating for shear in the longitudinal direction shall begin at a distance $^{h}/2$ from the centerline of the bearing or face of integral cross beams (h= total depth).

13.2.4 Concrete Decks

For all concrete bridge decks, except flat slab bridges, that are designed per current AASHTO criteria for HS-20 loading or heavier, loading will be considered structurally sufficient and need not be rated. However, for existing bridge decks having any of the following conditions, rating of the deck is required:

1. Deck was designed for live loads lighter than HS-20.
2. Deck overhang is more than half the girder spacing.
3. Bridge Inspection Report Code is 4 or below.
4. When the original traffic barrier(s) or rail have been replaced by heavier barrier.

When rating of the deck is required, live load shall include all vehicular loads as specified in Section 13.1.1.I. Live load moments for the HS20 truck shall be per Section 3.24.3.1 of the AASHTO *Standard Specifications*. Live load moments for the legal and overload trucks shall be per the AASHTO *Manual for Maintenance Inspection of Bridges*.

13.2.5 Concrete Crossbeams

Live loads can be applied to the crossbeam as moving point loads at any location between curbs that produce the maximum effect.

When rating for shear in crossbeams, current AASHTO *Design Specifications* requires shear design to be at the face of support if there is a concentrated load within a distance “d” from the face of support. This requirement is new relative to earlier editions of AASHTO *Design Specifications* that allowed shear reinforcement design to be at a distance “d” from the face of support. When rating existing crossbeams that show no indication of distress on the latest inspection report, but have a rating factor of less than one (1.0), a more detailed/accurate shear analysis should be performed. One acceptable method is the “Strut and Tie” model analysis. For existing box girders and T-beams integral with the crossbeams, in lieu of this detailed analysis, dead and live loads can be assumed as uniformly distributed and the shear rating performed at a distance “d” from the face of support.

13.2.6 In-Span Hinges

For in-span hinges, rating for shear and bending moment should be performed based on the reduced cross-sections at the hinge seat. Diagonal hairpin bars are part of this rating as they provide primary reinforcement through the shear plane.

13.2.7 Concrete Box Girder Structures

Bridges with spread box girders shall be rated on a per box basis. Otherwise, the rating shall be on the per bridge basis for all applied loads.

13.2.8 Prestressed Concrete Girder Structures

Rate on a per member basis.

13.2.9 Concrete Slab Structures

Rate cast-in-place solid slabs on a per foot of width basis. Rate precast panels on a per panel basis. Rate cast-in-place voided slabs based on a width of slab equal to the predominant center-to-center spacing of voids.

When rating flat slabs on concrete piling, assume pin-supports at the slab/pile interface of interior piers and the slab continuous over the supports. If ratings using this assumption are less than 1.0, the piles should be modeled as columns with fixity assumed at 10 feet below the ground surface.

Pile caps are to be rated if deemed critical by the engineer.

13.2.10 Steel Structures

On existing bridges, checking of fatigue and serviceability shall not be part of the rating evaluation.

13.2.11 Steel Floor Systems

Floorbeams and stringers shall be rated as if they are simply supported. Assume the distance from outside face to outside face of end connections as the lengths for the analysis. Live loads can be applied to the floorbeam as moving point loads at any location between curbs, which produce the maximum effect.

Rating of connections is not required unless there is evidence of deterioration.

13.2.12 Steel Truss Structures

The capacity of steel truss spans, regardless of length, shall follow the AASHTO *Guide Specifications for Strength Design of Truss Bridges* (Load Factor Design) and the AASHTO *Standard Design Specifications*. In the event the two specifications are contradictory, the guide specification shall be followed.

Rate on a per truss basis or perform a 3-D analysis or simplified distribution methods. Assume nonredundancy of truss members and pinned connections.

In general, rate chords, diagonals, verticals, end posts, stringers, and floorbeams. Do not rate connections unless there is evidence of deterioration, except for pinned connections with trusses. For pin-connected trusses, also analyze pins for shear, and the side plates for bearing capacity.

For truss members that have been heat-straightened three or more times, deduct 0.1 from ϕ(Phi).

13.2.13 Timber Structures

Unless the species and grade is known, assume Douglas fir, select structural for members installed prior to 1955 and Douglas fir, No. 1 after 1955. The allowable stresses for beams and stringers shall be as listed in the AASHTO *Standard Specifications*.

The nominal dimensions should be used to calculate dead load, and the net dimensions to calculate section modulus. If the member is charred, it may be assumed the $^{1}/_{4}$-inch of material is lost on all surfaces. Unless the member is notched or otherwise suspect, shear need not be calculated.

When calculating loads, no impact is assumed.

13.2.14 Widened or Rehabilitated Structures

For widened bridges, rate crossbeams in all cases.

For existing bridges, a load rating shall be performed if the load carrying capacity of the longitudinal members is altered, or the dead and live loads have increased due to the widening.

Longitudinal rating for the widened portion will be required only when the width of the widened portion on one side of the structure is greater than or equal to 10′-0” or more throughout the length of the structure.

For rehabilitated bridges, a load rating will be required if the load carrying capacity of the structure is altered by the rehabilitation.