OHIO DEPARTMENT OF TRANSPORTATION

CENTRAL OFFICE, 1980 W. BROAD ST., COLUMBUS, OHIO 43216-0899

January 19, 2018

To: Users of the Bridge Design Manual

From: Tim Keller, Administrator, Office of Structural Engineering

By: Sean Meddles, Assistant Administrator, Office of Structural Engineering

Re: 2018 First Quarter Revisions

Revisions have been made to the ODOT Bridge Design Manual, July 2007. These revisions shall be implemented on all Department projects that begin Stage 2 plan development date after January 19, 2018. Implementation of some or all of these revisions for projects further along the development process should be considered on a project-by-project basis.

This package contains the revised pages. The revised pages have been designed to replace the corresponding pages in the book and are numbered accordingly. Revisions, additions, and deletions are marked in the revised pages by the use of one vertical line in the right margin. The header of the revised pages is dated accordingly.

To keep your Manual correct and up-to-date, please replace the appropriate pages in the book with the pages in this package.

To ensure proper printing, make sure your printer is set to print in the 2-sided mode.

The July 2007 edition of the Bridge Design Manual may be downloaded at no cost using the following link:

http://www.dot.state.oh.us/Divisions/Engineering/Structures/Pages/default.aspx

Attached is a brief description of each revision.

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Summary of Revisions to the July 2007 ODOT BDM

BDM Section	Affected Pages	Revision Description
301.4.4.1	3-3.1	Updated LRFD references to 8th Edition.
301.4.4.2	3-3.2	Updated LRFD references to 8th Edition.
301.4.4.2.b	3-3.3	Updated LRFD references to 8th Edition.
301.5.3	3-5	The reference to Figure 301.5.3-4 was deleted. The contents of this figure have been incorporated into Figure 301.5.3-3.
302.2.2	3-13.1	The reference to shrinkage and temperature reinforcement was deleted, and replaced by a reference to Figures 302.2.2-2 and 302.2.2-3 for additional reinforcement in the underside of the deck overhang.
302.2.4.1	3-15	Updated LRFD references to 8th Edition.
302.2.5	3-16	The sides of the haunch for steel beam and girder structures are now required to be vertical and aligned with the edges of the top flange, similar to current practice for prestressed I-beam structures. This change was made in response to poor performance (spalling) of the tapered haunches previously used for steel beam and girder structures.
302.5.1.3	3-38	Updated LRFD references to 8th Edition.
302.5.2.2	3-41	Updated LRFD references to 8th Edition.
303.3.2.1.b	3-58	Updated LRFD references to 8th Edition.
303.3.2.8	3-61	Updated LRFD references to 8th Edition.
303.4.3	3-68	Updated LRFD references to 8th Edition.
Figures 301.5.3-1 through 301.5.3-3	--	These figures were revised in accordance with the latest AASHTO LRFD revisions to development and lap lengths.
Figure 301.5.3-4	--	This figure was deleted (contents revised and incorporated into Figure 301.5.3-3).
Figures 302.1.4.3-1 and 302.1.4.3-2	--	The haunches were revised to show vertical sides aligned with the edges of the top flange, consistent with 302.2.5.

BDM Section	Affected Pages	Revision Description
Figure 302.2.2-1	--	The following revisions were made to the Concrete Deck Design Aid chart: i. For some effective span lengths, the deck thickness was revised to be consistent with the formula for deck thickness given in 302.2.1. As a result of the deck thickness revisions, some changes were also made to reinforcing. ii. The chart was revised to be compatible with all beam sections shown on the current Standard Drawing PSID-1-13 and steel beams/girders with top flange widths ranging from 12” to 30”. iii. Cutoff lengths were revised to include bar development and to conform to the guidance given in ACI 318-14, R9.7.3.2 and R9.7.3.3. iv. Updated LRFD references to the 8th Edition.
Figures 302.2.2-2 through 302.2.2-3	--	These figures were revised to add guidance relating to the additional reinforcement in the underside of the deck overhang.
Figure 302.2.3-1	--	The haunches were revised to show vertical sides aligned with the edges of the top flange, consistent with 302.2.5.
Figure 302.2.5-1	--	The haunches were revised to show vertical sides aligned with the edges of the top flange, consistent with 302.2.5.
702.7	7-7	Note [702.7-1] was revised to reflect the use of vertical sides of haunches. The Designer Note was also revised to clarify that the note applies to both rolled beams and plate girders.
S4.6.4.3	10-10	Updated LRFD references to 8th Edition.
S5	10-11 to 10-14	Updated LRFD references to 8th Edition.

301.4 LOADING REQUIREMENTS

301.4.1 HIGHWAY BRIDGES

All bridges designed to carry highway traffic shall be designed for an HL-93 loading as specified in *LRFD 3.6.1.2.1* and a future wearing surface (FWS) of 60 psf [2.87 kPa].

301.4.2 PEDESTRIAN AND BIKEWAY BRIDGES

Pedestrian and bikeway bridges shall be designed in accordance with the latest edition of the AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges; ODOT design guidelines; and this Manual. ODOT's most current design guidelines are available at ODOT'S Office of Local Projects website, www.dot.state.oh.us/local/.

Where sidewalks, pedestrian, and/or bicycle bridges are intended to be used by maintenance and/or other incidental vehicles, an H15-44 [M13.5] vehicle, as shown in Figure 301.4.2-1, shall be included in the design loading. The H15-44 lane loading should not be considered. The vehicle live load shall not be placed in combination with the pedestrian live load and the dynamic load allowance need not be applied to the H15 vehicle.

301.4.3 RAILROAD BRIDGES

Facilities that are operated and maintained by the railroad shall be designed according to the specifications and design standards used by the railroad in its normal practice. Facilities that are operated and maintained by ODOT or other local agency shall conform to the AASHTO LRFD Bridge Design Specifications and this Manual.

301.4.4 SEISMIC DESIGN

Earthquakes arise from the movement of underlying bedrock. Ground motion resulting from the movement of underlying bedrock can be amplified or dampened by the overlying soil profile. Designers shall analyze soil borings to identify overlying soil profiles that can amplify ground motion propagating from underlying rock according to *LRFD 3.10.3.1*. Bridges located in Site Class D, E and F may require additional design considerations as noted in BDM Sections 301.4.4.1.

301.4.4.1 SEISMIC PERFORMANCE ZONE 1

All bridges in the State of Ohio are located within Seismic Performance Zone 1.

Bridges designed according to the Strength and Service Limit States of the *AASHTO LRFD Bridge Design Specifications* are assumed to have sufficient capacity to resist Seismic

Performance Zone 1 design loads applied at the Extreme Limit State. Seismic analysis is not required except as noted in BDM Sections 301.4.4.1.a and 301.4.4.1.b.

For bridges located in Site Class D, E and F, Designers shall determine the acceleration coefficient, S_{D1}, according to *LRFD Eq. 3.10.4.2-6* with F_V = 2.4. For Ohio, only areas where $S_1 \geq 0.042$ and Site Class D, E and F exist, will $0.10 \leq S_{D1} < 0.15$ (See Figure 301.4.4.1-1). For bridges founded at locations with $0.10 \leq S_{D1} < 0.15$, the transverse reinforcement requirements at the top and bottom of columns shall be as specified in *LRFD 5.11.4.1.4* and *5.11.4.1.5*. If sufficient geotechnical information is not available to determine Site Class, and the project is located in areas where $S_1 \geq 0.042$, then Designers shall assume $0.10 \leq S_{D1} < 0.15$. Otherwise, Designers shall assume $S_{D1} < 0.10$.

Designers may use the Seismic maps, *LRFD Figure 3.10.2.1-3* or the USGS US Seismic Design Maps web application to determine S_1 and S_{D1}.

301.4.4.1.a MINIMUM SUPPORT LENGTH REQUIREMENTS

To prevent the partial or complete collapse of the superstructure during seismic events, the bearing supports at the end of a superstructure unit shall be sized according to *LRFD 4.7.4.4*.

As a minimum, the overlapping distance of the superstructure and bearing areas shall meet 100% of the minimal support length, *N*, calculated according to *LRFD EQ. 4.7.4.4-1*. The minimum support lengths shall be measured normal to the centerline of supports. Designers shall account for expansion/contraction movements of the bearings when establishing final seat widths. Refer to Figure 301.4.4.1.a-1 for the application of support length requirements to a typical expansion elastomeric bearing at an abutment.

301.4.4.1.b HORIZONTAL CONNECTION FORCE

All structures shall have some mechanism to transfer horizontally applied superstructure loads (e.g. vehicular braking force, centrifugal force, vehicular collision force, friction load, water load, wind load, and wind load on live load) to the substructure to ensure structural stability. Examples of mechanisms include fixed bearings, bearing guides, abutment diaphragms, diaphragm guides, wing walls and wind locks. During a seismic event, these mechanisms that prevent the free lateral translation of the superstructure in any direction relative to the substructure, shall be designed to transfer an applied horizontal connection force at the Extreme Event I Limit State. Additional restraint for seismic loads (e.g. seismic pedestals) shall only be provided where the mechanisms noted above do not provide sufficient capacity. Refer to BDM Section 301.4.4.1.cfor bearing requirements.

The magnitude of the connection force shall be 0.15 or 0.25 times the tributary permanent load at the location of the restraint as determined in *LRFD 3.10.9.2*. The load factor for live load, γ_{EQ}, may be taken as 0.0. If sufficient geotechnical information is not available to determine Site Class, Designers shall assume the magnitude of the connection force is 0.25 times the tributary permanent load.

For restraint provided in multiple directions, *LRFD 3.10.8* applies.

The tributary permanent load defined in *LRFD 3.10.9.2* represents the factored dead load of the superstructure applying load to the device or object providing the directional restraint. If every bearing supporting the superstructure provides transverse restraint, the tributary permanent load applied to each restraint would equal the factored dead load reaction at each bearing. If only one transverse restraint was provided at each substructure unit, the tributary permanent load applied to each restraint would equal the sum of the factored dead load reactions for each bearing at the substructure unit. If only one transverse restraint was provided for the entire superstructure unit, the tributary permanent load applied to the restraint would equal the sum of the factored dead load reactions of every bearing. Longitudinal restraint connection forces would be determined similarly.

Because a structure in Seismic Performance Zone 1 is assumed to be able to carry the loads within the elastic strength range of its members, or is assumed to be properly detailed to prevent collapse beyond the elastic strength range of its members, analysis of the superstructure, substructure and foundation for the load effects resulting from the connection force is not required.

Crossframes to resist the horizontal connection force at the Extreme Event Limit state shall be provided to create a direct load path from the point of horizontal connection force application to the deck.

301.4.4.1.c REQUIREMENTS FOR BEARINGS

Unrestrained bearings that sustain irreparable damage during a seismic event are permissible provided loss of span is prevented by the design for the Horizontal Connection Force in BDM Section 301.4.4.1.b.

301.4.4.2 EXISTING STRUCTURES

Seismic vulnerability of a structure shall be considered for rehabilitation projects requiring complete deck or superstructure replacements. New substructure units shall be designed in accordance with *LRFD 3.10.9.2, 4.7.4.4* and *5.6.4.6*. If sufficient geotechnical information is not available, Designers may assume:

A. $A_S > 0.05$

B. $S_{D1} < 0.10$.

301.4.4.2.a SUPERSTRUCTURE

For projects where seismic vulnerability is considered, at bearing locations that will transmit the horizontal connection force from the substructure to the superstructure, crossframes designed to resist the horizontal connection force shall be provided to create a direct load path to the deck. For supports not in compliance with *LRFD 4.7.4.4*, seismic restrainers designed for the Horizontal Connection Force, specified in BDM Section 301.4.4.1.b, shall be provided.

301.4.4.2.b SUBSTRUCTURE

For projects where seismic vulnerability is considered, concrete columns at piers that transfer the seismic horizontal connection force, according to BDM Section 301.4.4.1.b, shall meet the spiral and tie ductility requirements of *LRFD 5.6.4.6*. Designers may consider releasing restraint provided by existing pier bearings as a viable seismic retrofit provided the abutments can accommodate the additional horizontal Strength and Service loadings. Otherwise, Designers shall provide the required confinement of the primary steel in the axially loaded substructure members.

One acceptable method to increase the amount of confinement provided in an existing concrete column is through the use of Fiber Reinforced Polymer (FRP) wrap systems. These systems are a viable alternative for dry columns supported on pile caps, spread footings and drilled shafts. Research has shown that providing a confining stress of 0.300 ksi in regions where plastic hinges may form at the top and bottom of columns as defined in *LRFD 5.11.4.1.5* and providing a confining stress of 0.150 ksi outside of the plastic hinge regions is sufficient to prevent buckling of the longitudinal reinforcement.

ODOT has a Proposal Note for Composite Fiber Wrap Systems which references the International Code Council Evaluation Service website (www.icc-es.org) for acceptable FRP wrap products. Refer to the Designer Notes for plan information associated with this work.

For bridges located in regions with an acceleration coefficient, $S_{D1} < 0.10$, Designers shall specify a confining stress due to FRP jacket (f_l) of 0.150 ksi for the entire height of the column from the top of the footing/drilled shaft to the bottom of the cap. For bridges located in regions with an acceleration coefficient, $S_{D1} \geq 0.10$, Designers shall specify a confining stress due to FRP jacket (f_l) of 0.300 ksi in the plastic hinge regions as defined in *LRFD 5.11.4.1.5* and 0.150 ksi in the remaining portions of the columns. The plans shall show an elevation view of the columns with these confining stress regions clearly defined.

301.4.5 APPLICATION OF LONGITUDINAL FORCES

For bearing types that permit rotation about the transverse axis of the bridge, all longitudinal load types shall be applied at the bearing elevation and moments resulting from eccentricity shall be ignored. The total factored longitudinal loading applied to the substructure at each expansion bearing shall not exceed the bearing's nominal (i.e. **unfactored**) resistance to longitudinal loading. Resistance in this instance is nominal because it is applied to the substructure as a loading.

splice length.

Reinforcing steel shall not project through expansion and contraction joints.

In lieu of lap splices, mechanical splices in accordance with the requirements of CMS 509 may be used. When specifying mechanical splices in congested areas, the Designer should consider staggering splice locations in order to meet the minimum spacing of reinforcing steel according to *LRFD 5.10.3.1*.

Designers shall use only mechanical type splices for #14 [43M] and #18 [57M] bars.

Splicing of reinforcing by welding is not permitted.

Where a horizontal construction joint is used in a column or pier, the reinforcement should be continuous and splices avoided if at all possible. An exception to this is the construction joint between a column and a footing, where the reinforcement should be discontinuous and adequate splice length should be furnished.

For tension splice lengths, see Figure 301.5.3-1.

For compression splice lengths, see Figure 301.5.3-2.

For development length requirements for reinforcing steel, see Figures 301.5.3-2 & 301.5.3-3.

301.5.4 CALCULATING LENGTHS AND WEIGHTS OF REINFORCING

Reinforcing steel lengths shall be calculated to the nearest 1 inch [25 mm]. Standard bend lengths shall be based on criteria in CMS 509.

The length or height of a spiral is defined as the distance out-to-out of coils, including the finishing turns at top and bottom.

The weight of the additional 1-1/2 coils of spiral required at the end by *LRFD 5.10.6.2* shall be calculated and included in the estimated quantities. For one, #4 [13M] spiral with a 4½" pitch, the weight, including the 1-1/2 coils at each end, is given by the following formula:

$$\text{Spiral Weight (lb)} = 0.148\,\pi H\sqrt{\left(\frac{4.5}{2}\right)^2 + (D - 0.5)^2} + 0.167\,\pi(D - 0.5)$$

Where: D = Outside Diameter of the Spiral (in)

H = Height or Length of the Spiral (ft)

$$\text{Spiral Weight (kg)} = 8.643\,\pi H\sqrt{\left(\frac{0.115}{2}\right)^2 + (D - 0.013)^2} + 0.248\,\pi(D - 0.013)$$

Where: D = Outside Diameter of the Spiral (m)

H = Height or Length of the Spiral (m)

See Figure 301.5.4-1 for area, weight and diameter of standard reinforcing. See Figure 301.5.4-2 for bar bending data. See Figure 301.5.4-3 for standard bar length deductions of common bends.

301.5.5 BAR LIST

Bar lists should include the following:

A. Bar Mark

B. Number of bars required

C. Overall length required of the bar

D. Total Weight for each bar mark

E. Column for type of bar:

   1. "ST" for straight
   2. "Number" assigned to
   3. "Numbered Bent Bar Detail"
   4. "Number" and "Series" for series bars

Dimensions are defined by letters A through Z associated with the “Numbered Bent Bar Detail” showing position of letters.

Spiral reinforcing shall also be included in the detail plan's bar list. The following information shall be shown on the bar list:

A. Core diameter

B. Pitch

C. Mark

D. Number

E. Height

F. Weight

G. Plan note for spiral bars

A sample bar list is provided in Figure 301.5.2-1.

301.5.6 USE OF EPOXY COATED REINFORCING STEEL

All reinforcing steel shall be epoxy coated except as noted for prestressed box beams in Section 302.5.1.8.

All approach slabs shall have epoxy coated reinforcing steel.

Concrete surfaces that include patches should be sealed with an epoxy-urethane sealer so the concrete color will remain uniform.

The designer should include in the plans actual details showing the position, location and area required to be sealed. A plan note should not be used to describe the location as there can be both description and interpretation problems.

The designer has the option to select a specific type of sealer, epoxy-urethane or non-epoxy. The designer may also use a bid item for sealer, with no preference, and allow the contractor to choose based on cost.

Due to poor performance, epoxy-only sealers shall not be used.

In areas where concrete surfaces have a history of graffiti vandalism, the designer may add a permanent graffiti coating meeting the requirements of Supplement 1083 on top of the epoxy-urethane or non-epoxy sealer. A plan note is available in BDM Section 600. The designer should limit the concrete surfaces that are treated with permanent graffiti coatings to those reachable by easy climbing and visible to the traveling public.

302.2 REINFORCED CONCRETE DECK ON LONGITUDINAL MEMBERS

302.2.1 DECK THICKNESS

For reinforced concrete decks on steel or concrete longitudinal members, the deck thickness shall be computed by the following formula:

Tmin (inches) = $(S + 17)(12) \div 36 \geq 8½''$

Tmin (mm) = $(S + 5200) \div 36 \geq 215$ mm

> Where: S is the effective span length in feet [millimeters] determined according to *LRFD 9.7.3.2*. Tmin shall be rounded up to the nearest one-quarter inch [5 mm].

The one inch [25 mm] wearing thickness, Section 302.1.3.1, is included in the minimum concrete deck thickness but should be excluded in the calculations for structural design of the deck slab.

302.2.2 CONCRETE DECK DESIGN

The concrete deck design shall be in conformance with the approximate elastic methods of analysis specified in the *AASHTO LRFD Bridge Design Specifications*, latest edition, and the additional requirements specified in this Manual. Refined methods of analysis and the empirical design method, *LRFD 9.7.2*, are prohibited. The design live load shall be HL-93 and the design dead load shall include an allowance for a future wearing surface equal to 0.06 k/ft^2.

Reinforcement shall be placed in the underside of the deck overhang as noted in Figures 302.2.2-2 and 302.2.2-3. The clear cover measured to the transverse steel shall be 1 ½”.

Deck designs for superstructures with effective span lengths ranging from 7.0 ft. to 14.0 ft. in 0.5 ft. increments are provided in Figures 302.2.2-1, 302.2.2-2 and 302.2.2-3. These designs apply for the full length of the bridge and preclude the need for additional transverse reinforcement at supported deck ends. The design of overhang reinforcement is valid for BR-1-13, SBR-1-13, BR-2-15 and TST-1-99 barrier systems. A complete list of design assumptions is

For stringer type bridges with reinforced concrete decks, the secondary bars shall be placed above the top of deck primary bars. This helps in reducing shrinkage cracking and adds additional cover over the primary bars.

For reinforced concrete deck slabs on non-composite stringer type bridges, where the main reinforcement is transverse to the stringers, additional top longitudinal reinforcement shall be provided in the negative moment region over the piers. This additional secondary reinforcement shall be equal to the distributional reinforcement (1/3 of the main reinforcement). This additional reinforcement shall be uniformly spaced and furnished in length equal to the larger of: 40 percent of the length of the longer adjacent stringer span or a length that meets the requirements of *LRFD 5.10.8.1.2c*.

For composite designs, the total longitudinal reinforcement over a pier shall meet the requirements of *LRFD 6.10.1.7*.

Additional negative moment reinforcement should be placed approximately symmetrical to the centerline of pier bearings but with every other reinforcing bar staggered 3 feet [1000 mm] longitudinally.

302.2.4.2 TRANSVERSE

To facilitate the placement of reinforcing steel and concrete in transversely reinforced deck slabs top and bottom main reinforcement shall be equally spaced and placed to coincide in a vertical plane.

For steel beam or girder bridges with a skew of less than 15 degrees the transverse reinforcing may be shown placed parallel to the abutments. Bridges with a skew greater than 15 degrees or where the transverse reinforcing will interfere with the shear studs should have the transverse reinforcement placed perpendicular to the centerline of the bridge. Refer to the appropriate Standard Bridge Drawing for the requirements on slab bridges.

For prestressed I-beams, transverse reinforcing shall be placed perpendicular to the centerline of the bridge.

For composite box beam decks, the transverse reinforcing steel may be placed parallel to the abutment.

For steel beam or girder bridges, the clearance of the bottom transverse bars over the top of any bolted beam splice plates or moment plates should be checked as reinforcing bars at a skew generally cannot be placed between bolt heads.

302.2.5 HAUNCHED DECK REQUIREMENTS

Concrete decks on steel beam, girder or prestressed I-beam structures shall have a concrete haunch to prevent a thinning of the deck slab as a result of unforeseen variations in beam camber. At a minimum, the design haunch shall allow for 2 inches [50 mm] of excessive camber. The sides of the haunch shall be vertical and shall be aligned with the edges of the top flange. See Figures 302.2.5-1 & 302.2.5-2.

302.2.6 STAY IN PLACE FORMS

Galvanized steel or any other material type, stay in place forms, shall not be used.

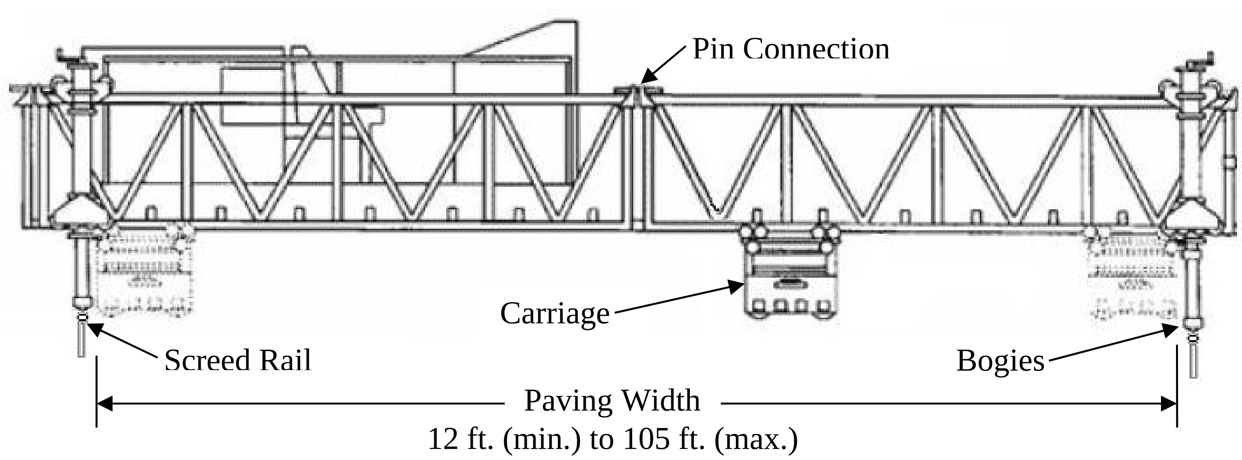
302.2.7 CONCRETE DECK PLACEMENT CONSIDERATIONS

Mechanized finishing machines are preferred to hand finishing methods for both consistency of surface finish and economics. Designers should be aware of finishing machine limitations in order to avoid deck designs that require hand finishing methods.

The placement of deck concrete using mechanized finishing machines alone does not ensure a smooth riding surface. Achieving a smooth riding surface as well as ensuring the proper geometry of the concrete deck is further complicated by deflections of the concrete falsework and of the main structural support members during the placement operation. The Contractor is responsible for designing falsework and finishing machine support to minimize deflection during placement, but the Designer is responsible for deflections induced by deck placement on the superstructure. Many complications due to deflection during placement can be avoided with proper design considerations.

302.2.7.1 FINISHING MACHINES

Mechanized finishing machines are comprised of fabricated truss sections pinned together to span the bridge deck width to be paved. The truss spans are supported at each end on a set of wheels, called "bogies," which ride along the length of the bridge on screed rails. Suspended below the truss is a finishing head, called a "carriage," which levels, compacts, vibrates and finishes the concrete.



302.5.1.2 STRANDS

Debonding of strands, by an approved plastic sheath, shall be done to control stresses at the ends of the beams. Refer to Section 302.5.2.2.d for debonding limits.

Deflecting of strands in box beams to limit stresses shall not be allowed.

The designer shall show on the plans the number, spacing and length of debonding. The box beam fabricator may have the option to change the position of debonding as long as the change is still symmetrical.

All strands extended from a beam to develop positive moment resistance shall not be debonded strands.

302.5.1.2.a TYPE, SIZE OF STRANDS

A. Low-relaxation ½ inch diameter (A_S = 0.153 in^2) seven wire uncoated strands, ASTM A416, Grade 270.

B. Low-relaxation ½ inch diameter (A_S = 0.167 in^2) seven wire uncoated strands, ASTM A416, Grade 270.

302.5.1.2.b SPACING

Strands shall be spaced at increments or multiples of 2 inches.

The location of the centerline of the first row of strands shall be 2 inches from the bottom of the beam. All strands shall be completely enclosed by the #4 stirrup bars. Strands near the top flange shall be placed below all transverse and longitudinal reinforcing steel and to the left and right of the void.

302.5.1.2.c STRESSES

Initial prestressing loads for low-relaxation strand shall be according to AASHTO requirements and shall be detailed on the plans.

Initial stress .... 0.75 f'_s = 202,500 psi

Initial tension load ....30,982 lb/strand (A_S = 0.153 in^2)

33,818 lb/strand (A_S = 0.167 in^2)

Initial stress ....0.75 f'_s = 1400 Mpa

Initial tension load ....138 600 N/strand (A_S = 99 mm^2)

151 200 N/strand (A_S = 108 mm^2)

302.5.1.3 COMPOSITE

Composite reinforced deck slabs on prestressed box beams shall be a minimum of 6 inches [155 mm] thick and shall be reinforced with #6 [#19M] bars. The longitudinal bars shall be spaced at 18" [450 mm] and the transverse bars spaced at 9" [225 mm]. For ease of placement on skewed structures, the transverse bars may be placed parallel to the substructure units with spacing measured parallel to the longitudinal axis of the structure.

On multiple span composite box beam bridges additional longitudinal reinforcing steel over the piers is required. The additional bars shall be alternately spaced with the standard longitudinal reinforcement and the pier bar's length shall be equal to the larger of: 40 percent of the length of the longer adjacent stringer span or a length that meets the requirements of *LRFD5.10.8.1.2c*. The pier bars should be placed longitudinally and approximately centered on the pier but with a 3 foot [1000 mm] stagger.

When designing the deck reinforcement for a multiple span structure, unless a more precise method of analysis is performed, the composite structure shall be conservatively modeled as a continuous beam on a single support centered on the pier.

Composite box beam structures with concrete parapets or sidewalks should not incorporate fit-up tolerances in the finished roadway width. To compensate for fit-up tolerances the composite deck and barrier and/or sidewalk should be designed to cantilever or overhang the boxbeam units by 2" [50 mm] to 8" [200 mm] each side with the fit-up being absorbed in the overhang. A mixture of 48" [1220 mm] and 36" [915 mm] boxbeam units may be necessary to meet this requirement.

See Figure 302.5.1.3-1 for a sketch of the cross-section of the composite deck superstructure.

mid-span for spans up to 50 feet [15 000 mm], at third points for spans from 50 feet [15 000 mm] to 75 feet [23 000 mm] and at quarter points for spans greater than 75 feet [23 000 mm].

302.5.2 I-BEAMS

AASHTO standard prestressed I-beam shapes, type II through type IV; modified type IV and WF36-49 through WF72-49 as shown in the standard bridge drawing, shall be used.

In designing prestressed I-beams, the non-composite section shall be used for computing stresses due to the beam and deck slab. The composite section shall be used for computing stresses due to the superimposed dead, railing and live loads. When designing the deck reinforcement for a multiple span structure, unless a more precise method of analysis is performed, the composite structure shall be conservatively modeled as a continuous beam on a single support centered on the pier.

Aside from access for shipping strands and clipped flanges as shown in the standard drawing, abrupt changes or discontinuities in the beam cross-section shall be avoided. If not, a refined analysis that accounts for section loss, resulting stress concentrations, and time dependent loading is required. Examples include providing breaks in the top flange to locate a utility or scupper for deck drainage.

302.5.2.1 DESIGN REQUIREMENTS

In order to prevent fabrication mistakes for beam length, the effect that the longitudinal grade has on dimensions measured along a beam’s length should be addressed in the plans. When the beam length measured along the grade differs from the beam length measured horizontally by more than 3/8" [10 mm], all affected dimensions measured along the length of the beam should be clearly labeled so that the fabricator can make the necessary allowances in the shop drawings. A Typical Detail note is available in Section 700.

When detailing beam elevations, dimension the locations of all inserts, hold-downs, etc. to the ends of the beam rather than the centerlines of bearing.

302.5.2.2 STRANDS

The preferred strand pattern is straight, parallel strands with no debonding. However, excessive tensile stresses may develop in the beam ends during the release of the prestressing force. To relieve these excessive stresses, the following strand patterns are allowed: (listed in order of preference)

A. Partially debonded bottom flange strands up to the limits specified in *LRFD 5.9.4.3.3*.

B. Combination of the maximum partially debonded bottom flange strands permitted by *LRFD 5.9.4.3.3* and draped strands.

Transforming strand area in order to increase section properties is not allowed.

302.5.2.2.a TYPE, SIZE

Use low-relaxation, 0.6-inch diameter (A_S = 0.217 in^2) seven wire uncoated strands, ASTM A416, Grade 270. The prestressing strand type and size shall be listed in the contract plan General Notes.

302.5.2.2.b SPACING

Strands shall be spaced at increments of 2 inches [50 mm].

A minimum 2 inch [50 mm] dimension from bottom of beam to centerline of the first row of strands and any exterior beam surface shall also be maintained.

302.5.2.2.c STRESSES

Initial prestressing loads for low-relaxation strand shall be as per AASHTO requirements and shall be detailed on the plans.

Initial stress .... 0.75 f'_s = 202,500 psi

Initial tension load..43,943 lb/strand (A_S = 0.217 in^2)

302.5.2.2.d DEBONDING

Debonding or shielding of the strands, with an approved plastic sheath, may be done at the beam ends to relieve excessive stresses. The following guidelines shall be followed for debonded strand designs:

A. The maximum debonded length at each end shall not be greater than 0.16L - 40". Where L equals the span length in inches.

B. A minimum of one-half the number of debonded strands shall have a debonded length equal to one-half times the maximum debonded length.

C. No more than 25% of the total number of strands in the I-beam shall be debonded.

D. No more than 40% of the strands in any row shall be debonded.

E. Debonded strands shall be symmetrical about the centerline of the beam.

The designer shall show on the detail plans the number, spacing and the length of required debonding per strand.

on bedrock.

The minimum width of footing supported by a drilled shaft is the diameter of the shaft.

Where piling is used to support free-standing piers, the distance between centers of outside piles, measured across the footing, generally shall be not less than one-fifth the height of the pier.

Widths greater than the above shall be provided if required for proper bearing area or to accommodate the required number of piles.

The height of the pier shall be measured from the bottom of the footing to the bridge seat.

For multiple span bridges with continuity over piers, where the height of pier is more than 50 percent of the length of superstructure from the point of zero movement to such pier, it may be assumed that the pier will bend or tilt sufficiently to permit the superstructure to expand or contract without appreciable pier stress. This assumption is not permissible if the piers are skewed more than 30 degrees. The above rule does not apply to rigid frame or arch bridges.

Slender columns of either concrete or steel may be designed to bend sufficiently to permit the superimposed superstructure to expand and contract, but the resulting bending stresses shall not exceed the allowable.

During phased construction of a capped pile pier, do not design a pier phase to be supported on less than three (3) piles. For cap and column piers, do not design a phase to be supported on less than two (2) columns.

For a new or replacement structure, individual free-standing columns without a cap are not permitted.

303.3.1.1 BEARING SEAT WIDTHS

Pier bearing seat widths for reinforced concrete slab bridges should conform to Standard Bridge Drawing CPP-1-08. Also see Section 303.3.2.5 of this Manual.

Pier caps on piles, drilled shafts or on columns are normally a minimum of 3'-0" [915 mm] wide. This is the standard width used for continuous span prestressed box beams and I-beams. Bearing seat widths of 3'-0" [915 mm], while normally adequate must be verified by the designer of the structure. Large bearings, skew angle, intermediate expansion devices, AASHTO earthquake seat requirements, etc. may require additional width.

303.3.1.2 PIER PROTECTION IN WATERWAYS

See Section 200 of this Manual for piling protection requirements and Section 600 for a plan note to be added to design drawings when the Capped Pile Pier Standard Bridge Drawing is not referenced.

303.3.2 TYPES OF PIERS

303.3.2.1 CAP AND COLUMN PIERS

303.3.2.1.a CAPS

When designing the cantilever portions of cap and column piers, the design moments shall be calculated at the actual centerline of the column.

The uppermost layers of longitudinal reinforcing steel in the pier cap shall not be lap spliced at the centerline of a column.

Cap dimensions should be selected to meet strength requirements and to provide necessary bridge seat widths according to BDM Section 301.4.4.1.a. Caps should be cantilevered beyond the face of the end column to provide approximately balanced factored dead load moments in the cap. The end of the cantilevered caps should be formed perpendicular to the longitudinal centerline of the cap to allow for uniform development lengths for the reinforcing steel. Cantilevered pier caps may have the bottom surface of the cantilever sloped upward from the column toward the end of the cap. Cantilevered caps may be eliminated for waterway crossing where debris removal access is an issue.

303.3.2.1.b COLUMNS

Columns shall be designed as compression members according to *LRFD 5.6.4* and *5.11*.

Round columns are preferred. The minimum column diameter is 36”.

303.3.2.2 CAP AND COLUMN PIERS ON PILES

Piers supported on piles shall have separate footings under each column.

Column piers shall have at least 4 piles per footing.

For grade separation structures, the top of the pier's footings should be a minimum of 1'-0" [300 mm] below the level of the bottom of the adjacent ditch. This applies even though the pier is

Situations that require stringers to be continuous through, and in the same plane with a steel pier cap or crossbeam should be avoided if at all possible.

Designers should review all weld details for possible fatigue problems. Consult the Office of Structural Engineering for assistance in this area.

303.3.2.7 POST-TENSIONED CONCRETE PIER CAPS

Where vertical clearance or geometric considerations require stringers to be continuous through and/or in the same plane as the pier cap, a post-tensioned concrete cap should be investigated as a first option in lieu of a steel pier cap. However, this is a non-redundant design, and, as specified in Section 301.2, these structure types require a concurrent detail design review to be performed by the Office of Structural Engineering.

303.3.2.8 T-TYPE PIERS

In the cap of a T-type pier, the top layer of reinforcing bars shall extend the full length of the cap and be turned down at the end face the necessary development length. The second layer of reinforcing steel shall extend into the stem of the pier at least the necessary development length plus the depth of the cantilever at its connection to the stem. Cap widths shall provide sufficient bearing seat width according to BDM Section 301.4.4.1.a.

Stems of T-type piers and wall type piers shall be designed as compression members according to *LRFD 5.6.4*. Wall type piers are characterized by the absence of clearly defined cap member.

Lateral ties shall be provided according to *LRFD 5.10.4.3*. For bridges located in areas where $0.10 \leq S_{D1} < 0.15$, transverse steel at the top and bottom of concrete compression members shall meet the requirements of *LRFD 5.11.4.1.4* and *5.11.4.1.5*.

303.3.2.9 PIER USE ON RAILWAY STRUCTURES

For clearance requirements see Section 200 of this Manual. Items listed in Section 200 are only general rules and vary from railroad to railroad. The designer shall confirm with the individual railroad the actual physical dimension and design requirements.

303.3.2.10 PIERS ON NAVIGABLE WATERWAYS

Piers in the navigation channel of waterways, unless protected from collision by an adequate fendering system, shall be designed to resist collision forces based on AASHTO Guide Specification for Vessel Collision Design of Highway Bridges.

303.3.2.11 PIER CAP REINFORCING STEEL STIRRUPS

Stirrups for concrete beams of constant depth, such as pier caps, should be detailed using either 2 “U” bars with the vertical legs long enough to furnish the required lap length or a single bar closed type stirrup with 135° bends at both ends of the rebar. The single bar closed type stirrup should only be selected when minimum required lap lengths cannot be provided with the “U” type stirrup. The corner with the 135° bends of the closed type stirrup should be placed in the compression zone of the concrete beam.

303.4 FOUNDATIONS

303.4.1 FOOTINGS

303.4.1.1 MINIMUM DEPTH OF FOOTINGS

Refer to BDM Section 202.2.3.1 for Spread Footing Elevation requirements.

Pile and drilled shaft footings shall be founded as follows:

A. For footings partially or totally located inside the plan view of the OHWM, the top of the footing shall be at least 1-ft below the Thalweg.

B. For footings located entirely outside the plan view of the OHWM:

   1. The top of footing shall be a minimum of 1-ft below the finished ground line. The top of footing shall be at least 1-ft below the bottom of any adjacent drainage ditch.

   2. The bottom of footing shall not be less than 4-ft below, measured normal to, the finished groundline.

303.4.1.2 FOOTING RESISTANCE TO HORIZONTAL FORCES

For spread footings, if the frictional or shearing resistance of the supporting material as specified in *LRFD 10.6.3.4* is inadequate to withstand the horizontal forces, one or more of the following means, listed in order of preference, shall provide additional resistance:

A. Increase the footing width.

B. Use footing key and utilize the passive pressure acting on the key. Keys should be located within the middle-half of the footing width.

C. Use footing struts, sheeting or anchors .

Pier 5 ~

52 - 14" C.I.P. Reinforced Concrete Piles

36 piles installed vertical & 16 piles battered

Ultimate Bearing Value = 270 kip

Estimated Length = 85 ft

Order Length = 90 ft (Total Length = 4680 ft)

The difference in Ultimate Bearing Value between piers 1, 2, 3 & 4 and pier 5 requires 1 dynamic testing item.

Forward Abutment ~

30 - 12" C.I.P. Reinforced Concrete Piles

20 piles installed vertical & 10 piles battered

Ultimate Bearing Value = 152 kip

Estimated Length = 75 ft

Order Length = 80 ft (Total Length = 2400 ft)

No additional dynamic load testing items are required.

For this example, the Designer should include notes **[606.2-1]**, **[606.2-4]** and **[606.2-5]** from Section 606.2 in the General Notes. Note **[606.2-1]** should be modified as follows:

PILE DESIGN LOADS (ULTIMATE BEARING VALUE): The Ultimate Bearing Value is 152 kip per pile for the rear and forward abutment piles. The Ultimate Bearing Value is 250 kip per pile for Pier 1, 2, 3, and 4 piles and 270 kip per pile for Pier 5 piles.

Abutment Piles:

30 piles 70 ft long, order length (Rear)

30 piles 80 ft long, order length (Forward)

1 dynamic load testing item

Pier 1, 2, 3, and 4 Piles:

320 piles 75 ft long, order length

1 static load test item

1 subsequent static load test item

4 dynamic load-testing items

4 restrike items

Pier 5 Piles:

52 piles 90 ft long, order length

1 dynamic load testing item

The Designer should provide the following items in the Estimated Quantities:

Item	Extension	Total	Unit	Description
506	11100	Lump	Sum	Static Load Test
506	12200	1	Each	Subsequent Static Load Test
507	00500	4200	ft	12" Cast-In-Place Reinforced Concrete Piles, Driven
507	00550	4500	ft	12" Cast-In-Place Reinforced Concrete Piles, Furnished
507	00600	26,820	ft	14" Cast-In-Place Reinforced Concrete Piles, Driven
507	00650	28,680	ft	14" Cast-In-Place Reinforced Concrete Piles, Furnished
523	20000	6	Each	Dynamic Load Testing
523	20500	4	Each	Restrike

303.4.3 DRILLED SHAFTS

When determining the structural capacity of drilled shafts, multiply the compressive strength provided by the concrete mix design by a factor of 0.9 [e.g. f'_c = 0.9 (4.5 ksi) = 4.0 ksi for Class QC5] for all limit states.

To allow for the misalignment of drilled shafts that support single pier columns, the shaft diameter shall be 6 in. [150 mm] larger than the column diameter. To allow for misalignment of shafts into footings, footing widths shall be at least 1’-0” [305 mm] larger than the shaft diameter.

The diameter of bedrock sockets for drilled shafts are generally 6 in. [150 mm] less than the diameter of the shaft above the bedrock elevation. This downsize provides sufficient room is the shaft for the rock core barrel. Reinforcing steel cages should be based on the bedrock socket diameter.

For un-cased or temporarily cased drilled shafts, the spiral reinforcement should be a #4 [#13M] bar with a 4½ in. [115 mm] pitch. (Note: the above requirement shall be met even if the 4½ in. [115 mm] pitch may not meet the spiral requirements of *LRFD 5.6.4.6*) For shaft diameters 4.0 ft. and less, the out-to-out spiral diameter shall be 6 in. [150 mm] less than the rock socket diameter. For shaft diameters greater than 4.0 ft., the out-to-out spiral diameter shall be 12 in. [300 mm] less than the rock socket diameter. When steel casing is left in place, the spiral reinforcing pitch shall be 12 in. [300 mm].

The minimum clear distance between longitudinal and lateral reinforcement shall not be less than 5 times the maximum aggregate size. Where heavy reinforcement is required, consideration may be given to an inner and outer reinforcing cage.

Tension Lap Splice Lengths for Epoxy-Coated Bars (in.)

	1.5" Clear Cover			2" Clear Cover			2.5" Clear Cover			3" Clear Cover		
Bar Size	A	B	C	A	B	C	A	B	C	A	B	C
4	30	23	3.50	30	23	4.50	30	23	5.50	30	23	6.50
5	40	36	3.63	37	29	4.63	37	29	5.63	37	29	6.63
6	48	43	3.75	48	43	4.75	44	34	5.75	44	34	6.75
7	63	56	3.88	56	50	4.88	56	50	5.88	52	40	6.88
8	80	71	4.00	64	57	5.00	64	57	6.00	59	45	7.00
9	99	87	4.13	79	70	5.13	72	64	6.13	72	64	7.13
10	121	107	4.27	98	86	5.27	82	73	6.27	81	72	7.27
11	144	127	4.41	117	104	5.41	99	88	6.41	90	80	7.41

Notes:

A = Tension lap splice length for horizontal reinforcement, placed such that more than 12" of fresh concrete is cast below the reinforcement, and with c/c bar spacing greater than or equal to "C".

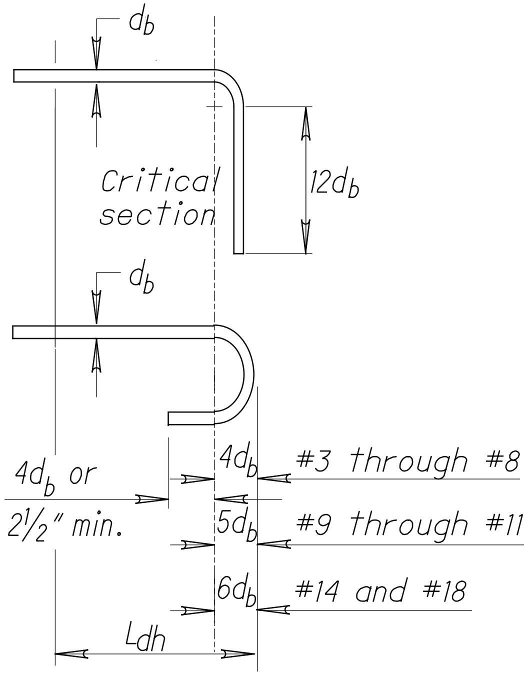
B = Tension lap splice length for vertical reinforcement or horizontal reinforcement, placed such that no more than 12" of fresh concrete is cast below the reinforcement, and with c/c bar spacing greater than or equal to "C".

C = Critical c/c bar spacing. If actual c/c bar spacing is less than critical c/c bar spacing, then do not use the table above. Calculate lap splice lengths per LRFD 5.10.8.2.1a and 5.10.8.4.3a.

1. Values shown are for Class B lap splices per LRFD 5.10.8.4.3a with f'c = 4 ksi and Fy = 60 ksi.
2. The table above applies to normal-weight concrete. If light-weight concrete is used, then calculate lap splice lengths per LRFD 5.10.8.2.1a and 5.10.8.4.3a.
3. The table above assumes that Atr = 0, where Atr is the cross-sectional area of transverse reinforcement which crosses the potential plane of splitting. If such transverse reinforcement is present, and the designer wishes to account for this, then calculate lap splice lengths per LRFD 5.10.8.2.1a and 5.10.8.4.3a. One example of such reinforcement is shear stirrups.
4. Where reinforcement is in excess of that required by analysis, the values shown in the table above may be reduced per LRFD Eq. 5.10.8.2.1c-4.

Figure 301.5.3-1

BAR NO.	DEVELOPMENT LENGTH FOR EPOXY-COATED STANDARD HOOKS IN TENSION ②		
	MOD. FACTOR / BASIC LENGTH, Lhb	CONCRETE COVER (LRFD 5.10.8.2.4b) (1.2) 0.8	TIES OR STIRRUPS (LRFD 5.10.8.2.4b) (LRFD 5.10.8.2.4c) (1.2) 0.8
4	10	10	10
5	12	12	12
6	14	13	13
7	17	16	16
8	19	18	18
9	21	20	20
10	24	23	23
11	27	26	26
14	32	—	—
18	43	—	—



HOOKED-BAR DETAILS FOR DEVELOPMENT OF STANDARD HOOKS

BAR NO.	COMPRESSION LAP SPLICES (LRFD 5.10.8.4.5a) STANDARD	WITH SPIRALS	WITH TIES
4	15	12	12
5	19	14	16
6	23	17	19
7	26	20	22
8	30	23	25
9	34	25	28
10	38	29	32
11	42	32	35

1. VALUES SHOWN ARE FOR NORMAL-WEIGHT CONCRETE WITH $f'c$ = 4 KSI AND Fy = 60 KSI. IF LIGHT-WEIGHT CONCRETE IS USED, THEN CALCULATE DEVELOPMENT LENGTH FOR STANDARD HOOKS IN TENSION PER LRFD 5.10.8.2.4.

2. FOR UN-COATED REINFORCING STEEL, VALUES SHALL BE DIVIDED BY 1.2.

Figure 301.5.3-2

Development Lengths for Epoxy-Coated Bars in Tension (in.)

	1.5" Clear Cover			2" Clear Cover			2.5" Clear Cover			3" Clear Cover		
Bar Size	A	B	C	A	B	C	A	B	C	A	B	C
4	23	18	3.50	23	18	4.50	23	18	5.50	23	18	6.50
5	31	27	3.63	29	22	4.63	29	22	5.63	29	22	6.63
6	37	33	3.75	37	33	4.75	34	26	5.75	34	26	6.75
7	49	43	3.88	43	38	4.88	43	38	5.88	40	31	6.88
8	62	54	4.00	49	44	5.00	49	44	6.00	45	35	7.00
9	76	67	4.13	61	54	5.13	56	49	6.13	56	49	7.13
10	93	82	4.27	75	67	5.27	63	56	6.27	63	55	7.27
11	111	98	4.41	90	80	5.41	76	67	6.41	70	61	7.41

Notes:

A = Development length for horizontal reinforcement, placed such that more than 12" of fresh concrete is cast below the reinforcement, and with c/c bar spacing greater than or equal to "C".

B = Development length for vertical reinforcement or horizontal reinforcement, placed such that no more than 12" of fresh concrete is cast below the reinforcement, and with c/c bar spacing greater than or equal to "C".

C = Critical c/c bar spacing. If actual c/c bar spacing is less than critical c/c bar spacing, then do not use the table above. Calculate development lengths per LRFD 5.10.8.2.1a.

1. Values shown are for f'c = 4 ksi and Fy = 60 ksi.
2. The table above applies to normal-weight concrete. If light-weight concrete is used, then calculate development lengths per LRFD 5.10.8.2.1a.
3. The table above assumes that Atr = 0, where Atr is the cross-sectional area of transverse reinforcement which crosses the potential plane of splitting. If such transverse reinforcement is present, and the designer wishes to account for this, then calculate development lengths per LRFD 5.10.8.2.1a. One example of such reinforcement is shear stirrups.
4. Where reinforcement is in excess of that required by analysis, the values shown in the table above may be reduced per LRFD Eq. 5.10.8.2.1c-4.

Figure 301.5.3-3

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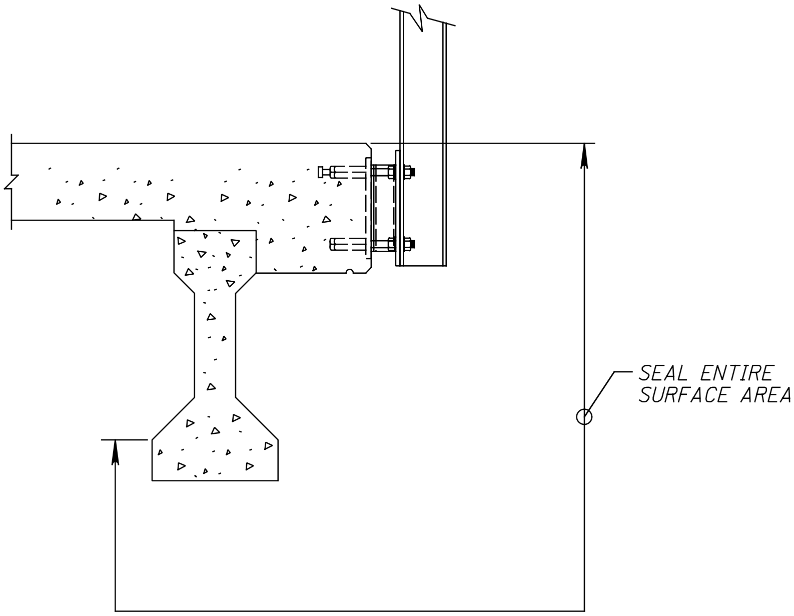
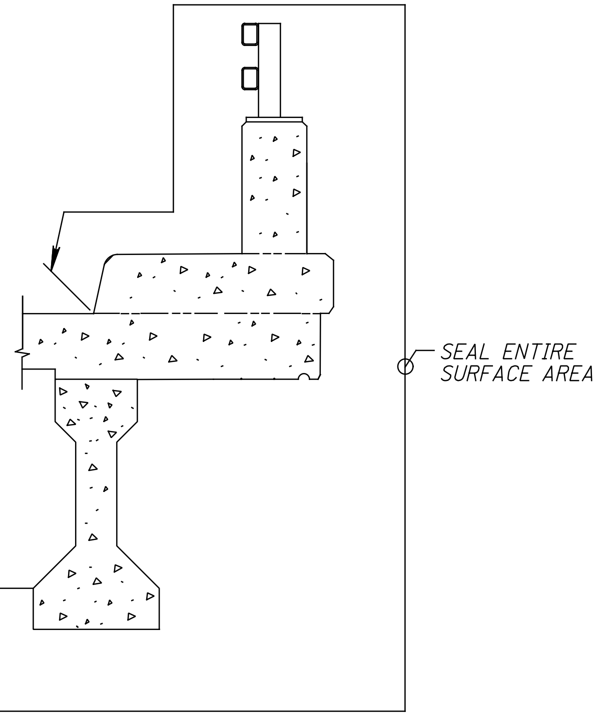
STD. BAR LENGTH DEDUCTIONS FOR COMMON BENDS, (IN)								
BAR NO.	STANDARD BEND (DEGREES)							
	D	45	D	90	D	135	D	180
#3	1 1/2"	1/4"	2 1/4"	1"	1 1/2"	1"	2 1/4"	1 7/8"
#4	2"	1/4"	3"	1"	2"	1 1/4"	3"	2 1/2"
#5	2 1/2"	3/8"	3 3/4"	1 1/2"	2 1/2"	1 5/8"	3 3/4"	3 1/8"
#6	3"	3/8"	4 1/2"	2"	3"	2"	4 1/2"	3 3/4"
#7	3 1/2"	1/2"	5 1/4"	2"	3 1/2"	2 1/4"	5 1/4"	4 3/8"
#8	4"	1/2"	6"	2 1/2"	4"	2 1/2"	6"	5"
#9	6 3/8"	5/8"	9 1/2"	3 1/2"	6 3/8"	3 3/8"	9 1/2"	6 7/8"
#10	7 1/8"	3/4"	10 3/4"	4"	7 1/8"	3 3/4"	10 3/4"	7 3/4"
#11	8"	3/4"	12"	4"	8"	4 1/4"	12"	8 5/8"
#14	12 1/8"	1"	18 1/4"	6"	12 1/8"	5 5/8"	18 1/4"	12"
#18	16"	1 3/8"	24"	8"	16"	7 1/2"	24"	15 3/4"

NOTE:
"D" IS THE DIAMETER OF THE BEND PER CONSTRUCTION AND MATERIAL SPECIFICATIONS ITEM 509.05

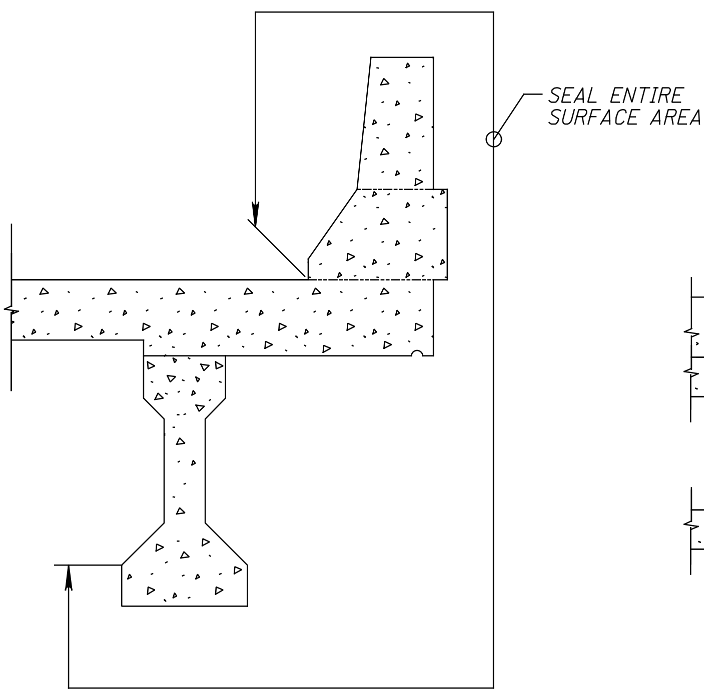
Figure 301.5.4-3



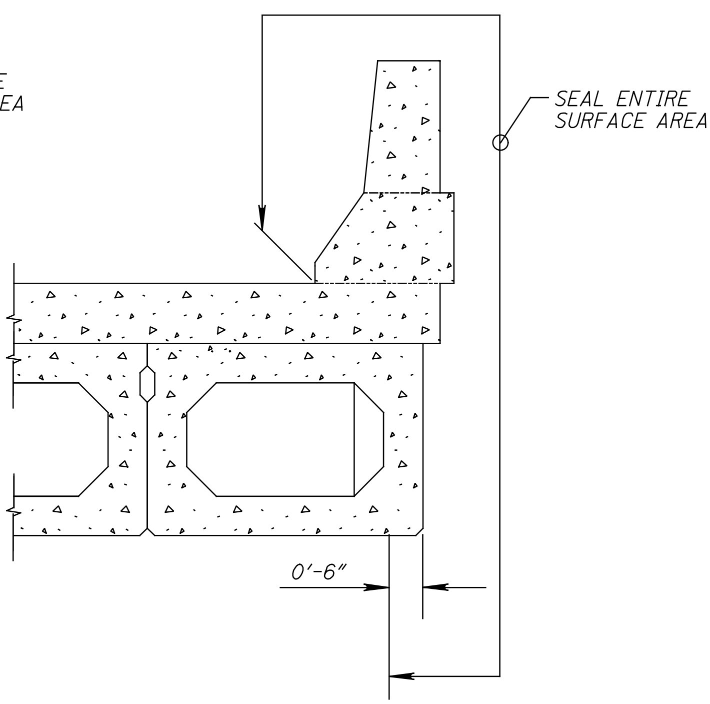
CONCRETE DECKS WITH
OVER THE SIDE DRAINAGE



CONCRETE DECKS WITH CURBS,
SIDEWALKS AND PARAPET



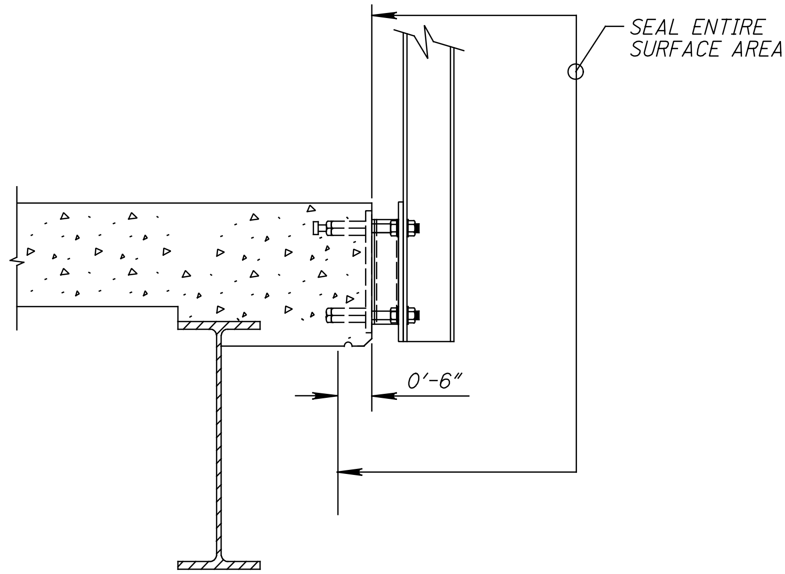
CONCRETE DECK WITH
DEFLECTOR PARAPET



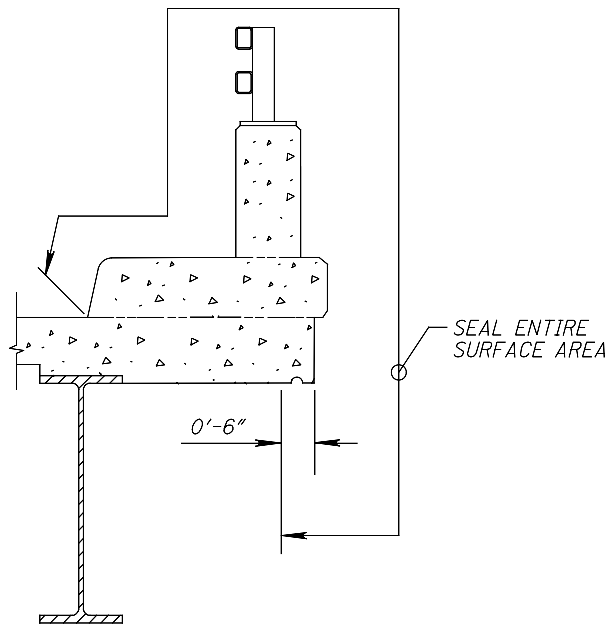
PRESTRESSED BOX BEAM DECK
WITH DEFLECTOR PARAPET

SEALING OF CONCRETE SURFACES, SUPERSTRUCTURE

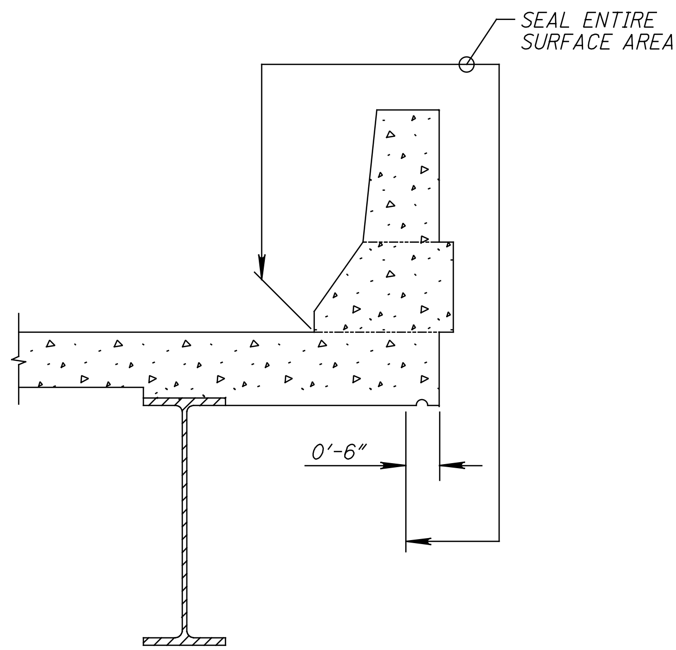
Figure 302.1.4.3-1



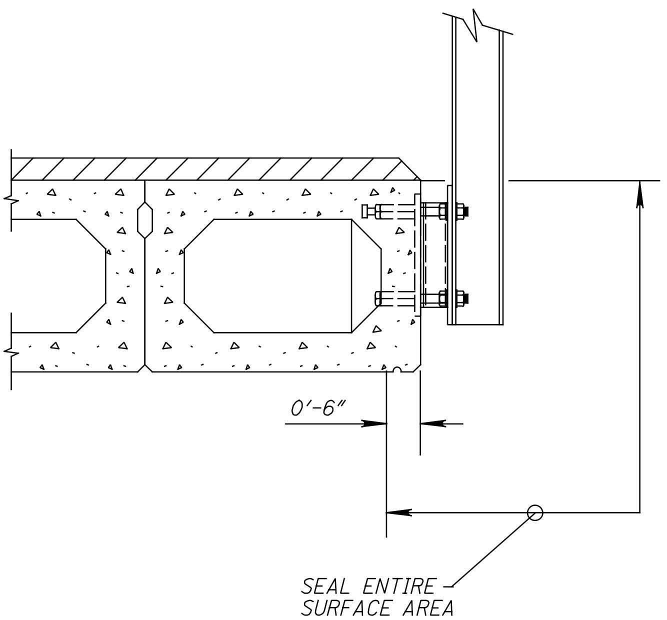
CONCRETE DECKS WITH
OVER THE SIDE DRAINAGE



CONCRETE DECKS WITH CURBS,
SIDEWALKS AND PARAPET



CONCRETE DECK WITH
DEFLECTOR PARAPET



PRESTRESSED BOX BEAM DECK
WITH OVER THE SIDE DRAINAGE

SEALING OF CONCRETE SURFACES, SUPERSTRUCTURE

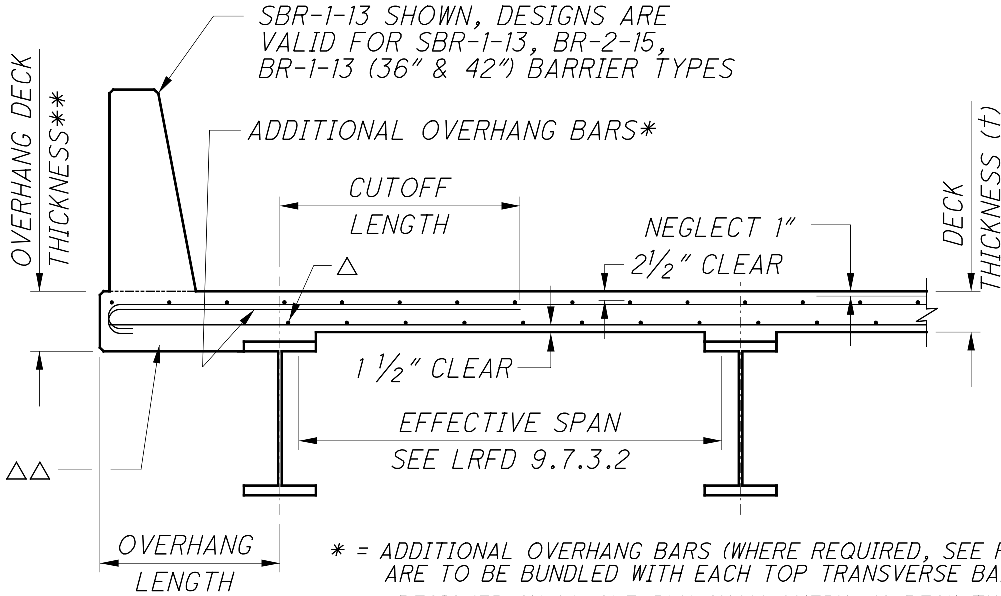
Figure 302.1.4.3-2

Concrete Deck Design Aid

Effective Span Length (ft.)	Deck Thickness (in.)	Overhang Deck Thickness (in.) (see Note 2k)	Transverse Steel						Longitudinal Steel			
			Top Bars				Bottom Bars		Top Bars		Bottom Bars	
			Size	Spa. (in.)	Additional Overhang Bar Size	Cutoff Length (in.)	Size	Spa.	Size	Spa.	Size	Spa.
7.0	8.50	10.50	#5	6.50	#5	54.0	#5	6.50	#4	12.50	#5	10.75
7.5	8.50	10.50	#5	6.25	#5	54.0	#5	6.25	#4	12.00	#5	10.25
8.0	8.50	10.50	#5	6.00	#5	54.0	#5	6.00	#4	11.50	#5	9.75
8.5	8.50	10.50	#5	5.75	#4	54.0	#5	5.75	#4	11.00	#5	9.25
9.0	8.75	10.75	#5	5.75	#4	54.0	#5	5.75	#4	11.00	#5	9.25
9.5	9.00	11.00	#5	5.75	#4	54.0	#5	5.75	#4	11.00	#5	9.25
10.0	9.00	11.00	#5	5.25	#4	48.0	#5	5.25	#4	10.00	#5	8.75
10.5	9.25	11.25	#5	5.25	#4	48.0	#5	5.25	#4	10.00	#5	8.75
11.0	9.50	11.50	#5	5.00	#4	48.0	#5	5.00	#4	9.50	#5	8.75
11.5	9.50	11.50	#6	5.75	#4	28.0	#5	5.75	#4	7.75	#5	8.75
12.0	9.75	11.75	#6	5.75	#4	28.0	#5	5.75	#4	7.75	#5	8.75
12.5	10.00	12.00	#6	5.75	--	--	#5	5.75	#4	7.75	#5	8.75
13.0	10.00	12.00	#6	5.75	--	--	#5	5.75	#4	7.75	#5	8.75
13.5	10.25	12.25	#6	5.75	--	--	#5	5.75	#4	7.75	#5	8.75
14.0	10.50	12.50	#6	5.75	--	--	#5	5.75	#4	7.75	#5	8.75

Notes:
1. Designs in accordance with AASHTO LRFD Bridge Design Specifications and the 2007 ODOT Bridge Design Manual
2. Design Assumptions:
   a. Four or more beam/girder lines
   b. Maximum beam/girder spacing = 15 ft. c/c
   c. Transverse steel is placed perpendicular to beam/girder lines
   d. Normal weight concrete with f′c = 4.5 ksi
   e. Reinforcing steel with fy = 60 ksi
   f. Monolithic Wearing Surface = 1.0 in.
   g. Future Wearing Surface = 0.06 ksf
   h. *LRFD 5.6.7* - Exposure Factor (γ_e) = 0.75
   i. Top cover = 2.50 in.; Bottom cover = 1.50 in.
   j. Maximum overhang width = 4.0 ft. (measured from cl. of fascia beam/girder to deck edge)
   k. Valid for BR-1-13 (36” & 42”), SBR-1-13, BR-2-15, and TST-1-99 barrier systems. The minimum overhang deck thickness for the TST-1-99 railing system is 18”.
3. Calculate Effective Span Length according to *LRFD 9.7.3.2* and round up to the nearest 0.5 ft. increment
4. Minimum Deck Thickness in accordance with BDM Section 302.2.1
5. Cutoff Length = length beyond the centerline of the fascia beam/girder where additional overhang bars are no longer required for strength and development.
6. Longitudinal bar spacing does not include additional reinforcing required for negative moments in accordance with *LRFD 5.6.3.2* (for prestressed beams) and *LRFD 6.10.1.7* (for steel beams/girders)
7. Refer to Figures 302.2.2-2 & 302.2.2-3 for more information

Figure 302.2.2-1



* = ADDITIONAL OVERHANG BARS (WHERE REQUIRED, SEE Figure 302.2.2-1) ARE TO BE BUNDLED WITH EACH TOP TRANSVERSE BAR.

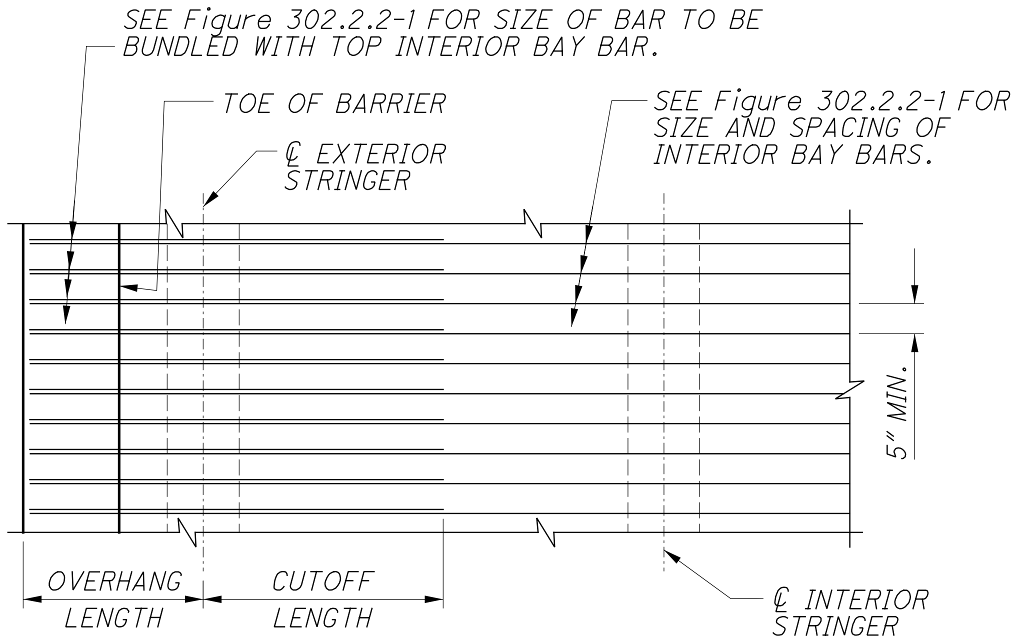
** = DESIGNER SHALL CLEARLY SHOW OVERHANG DECK THICKNESS ON PLANS. MINIMUM OVERHANG THICKNESS = t + 2"

△ = DO NOT PLACE LONGITUDINAL BARS IN THIS MAT BEYOND THE OUTER FLANGE TIP OF THE FASCIA BEAM/GIRDER

△△ = PROVIDE ADDITIONAL TRANSVERSE AND LONGITUDINAL REINFORCING STEEL IN THE BOTTOM OF THE DECK SLAB OVERHANG. IN THE TRANSVERSE DIRECTION, PROVIDE #4 BARS SPACED AT 2 TIMES THE TRANSVERSE BAR SPACING SHOWN IN Figure 302.2.2-1. IN THE LONGITUDINAL DIRECTION, PROVIDE THE SAME BAR SIZE AND SPACING AS SHOWN FOR LONGITUDINAL BOTTOM BARS IN Figure 302.2.2-1. THE CLEAR COVER MEASURED TO THE TRANSVERSE BARS SHALL BE 1 ½".

CUTOFF LENGTH = LENGTH BEYOND THE CENTERLINE OF THE FASCIA BEAM/GIRDER WHERE ADDITIONAL OVERHANG BARS ARE NO LONGER REQUIRED FOR STRENGTH AND DEVELOPMENT (SEE Figure 302.2.2-1)

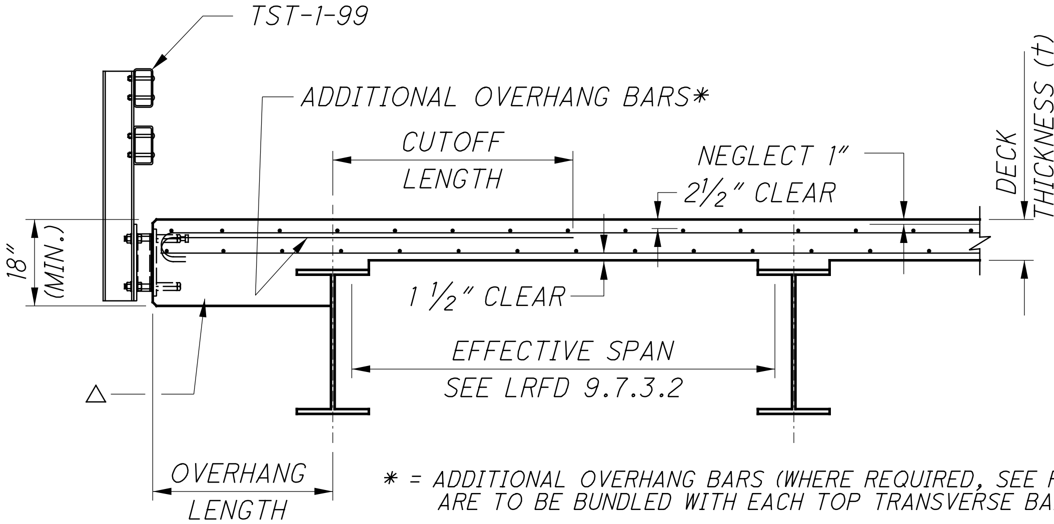
DECK SECTION



OVERHANG PLAN VIEW

TYPICAL DECK DETAILS-- ODOT LRFD STANDARD DECK DESIGN

Figure 302.2.2-2

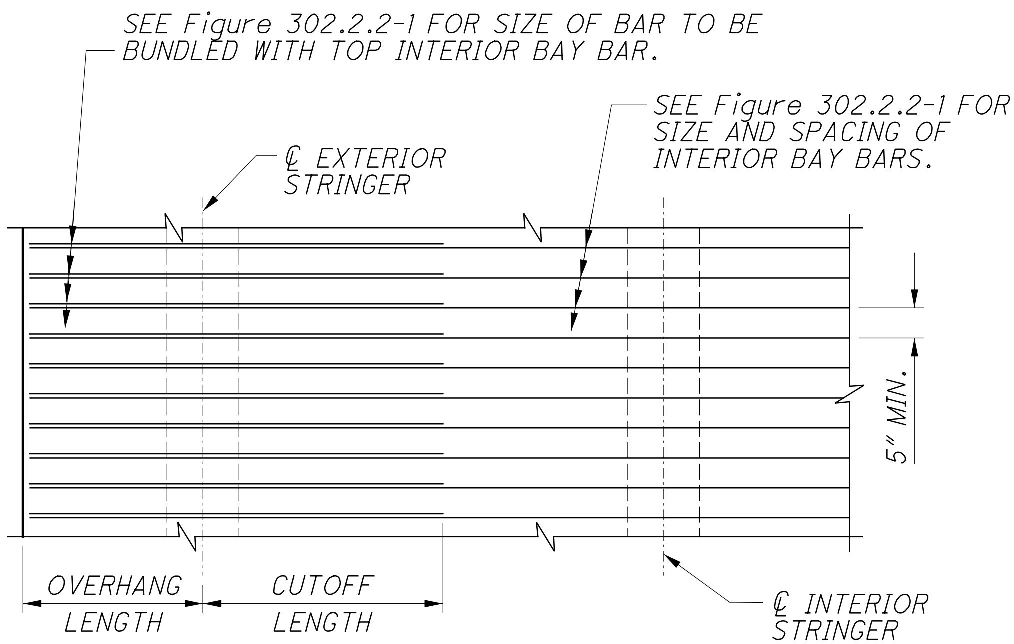


* = ADDITIONAL OVERHANG BARS (WHERE REQUIRED, SEE Figure 302.2.2-1) ARE TO BE BUNDLED WITH EACH TOP TRANSVERSE BAR.

△ = PROVIDE ADDITIONAL TRANSVERSE AND LONGITUDINAL REINFORCING STEEL IN THE BOTTOM OF THE DECK SLAB OVERHANG. IN THE TRANSVERSE DIRECTION, PROVIDE #4 BARS SPACED AT 2 TIMES THE TRANSVERSE BAR SPACING SHOWN IN Figure 302.2.2-1. IN THE LONGITUDINAL DIRECTION, PROVIDE #4 BARS SPACED AT 12" MAXIMUM. THE CLEAR COVER MEASURED TO THE TRANSVERSE BARS SHALL BE 1½".

CUTOFF LENGTH = LENGTH BEYOND THE CENTERLINE OF THE FASCIA BEAM/GIRDER WHERE ADDITIONAL OVERHANG BARS ARE NO LONGER REQUIRED FOR STRENGTH AND DEVELOPMENT (SEE Figure 302.2.2-1)

DECK SECTION



OVERHANG PLAN VIEW

TYPICAL DECK DETAILS-- ODOT LRFD STANDARD DECK DESIGN

Figure 302.2.2-3

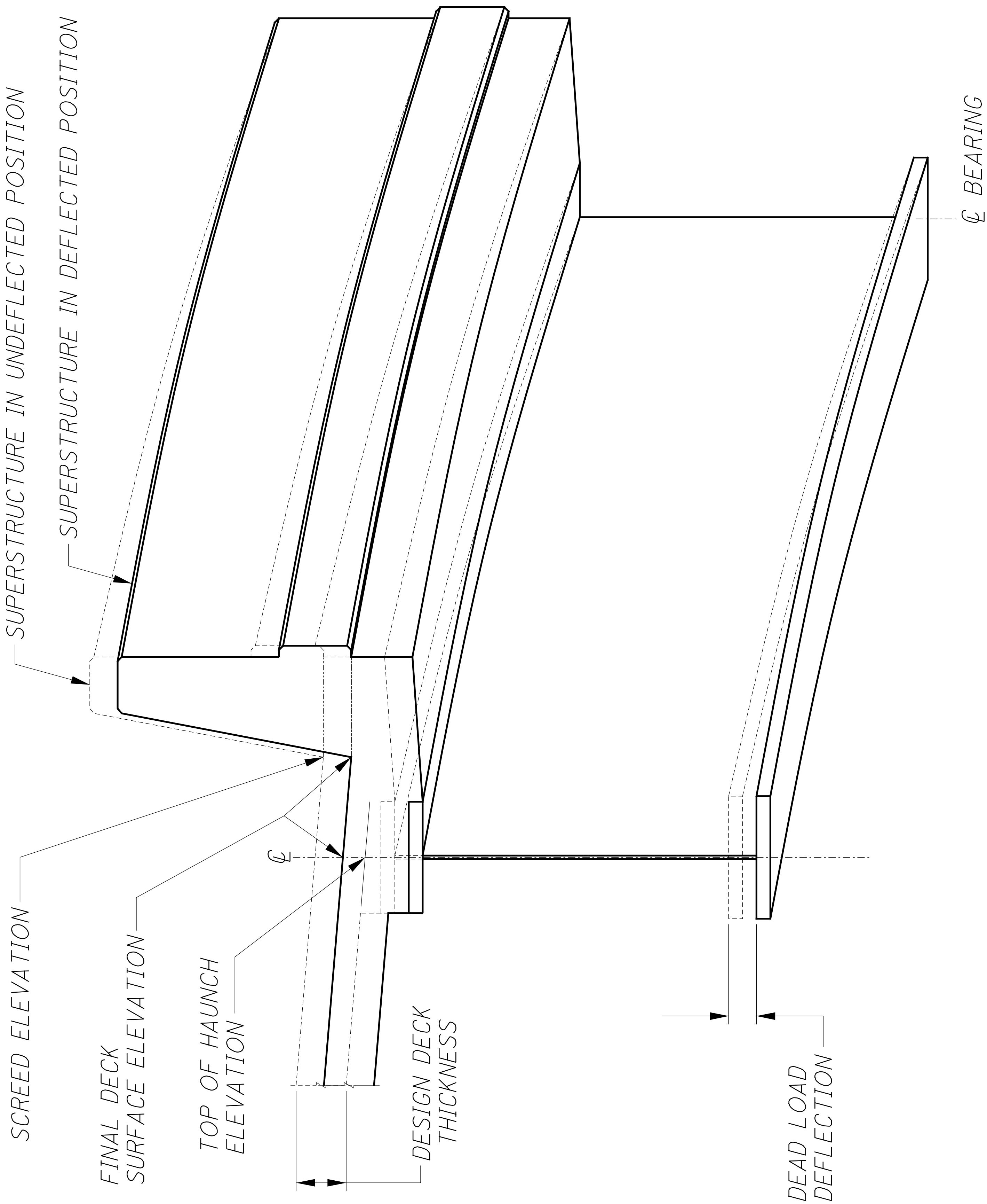


Figure 302.2.3-1

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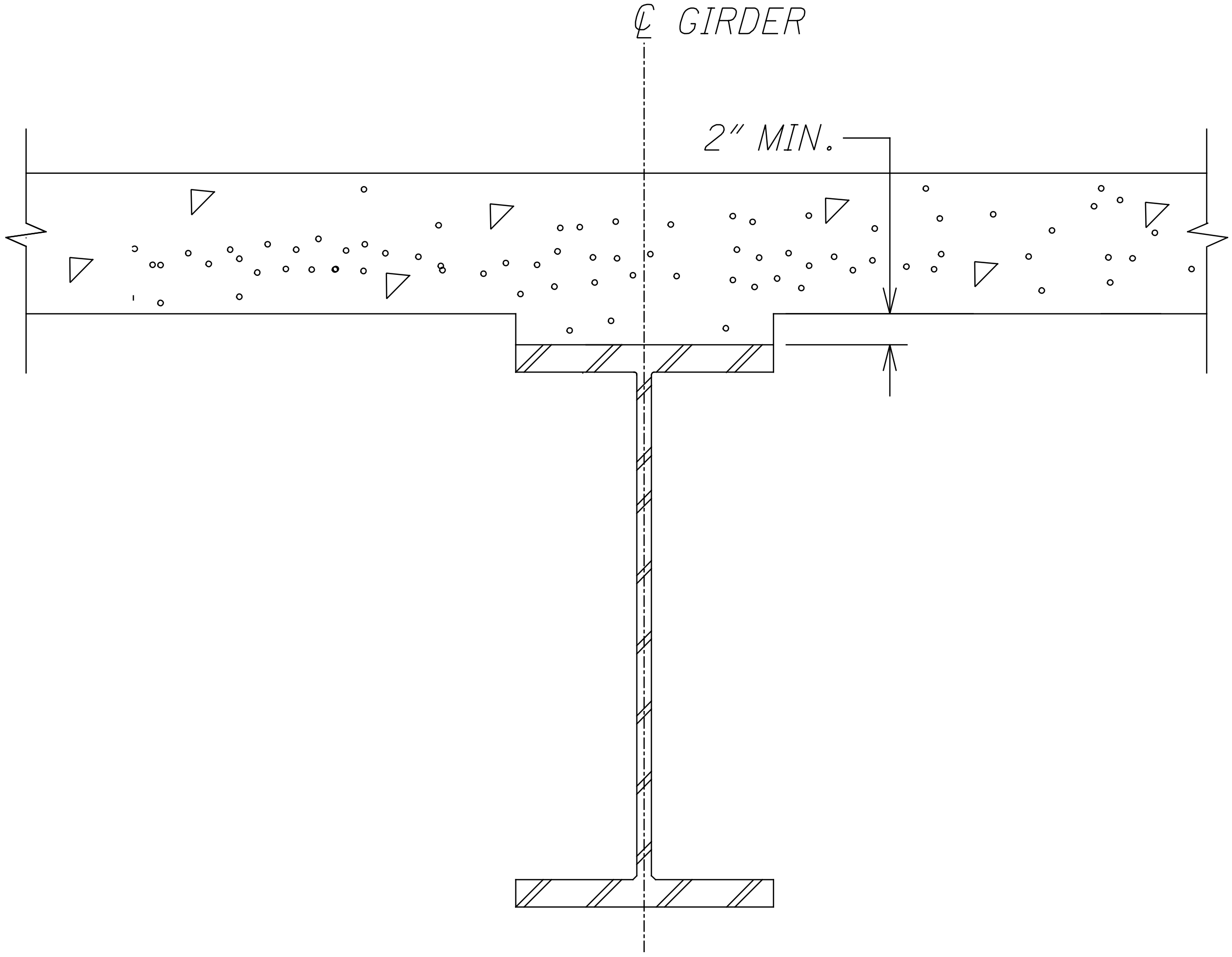
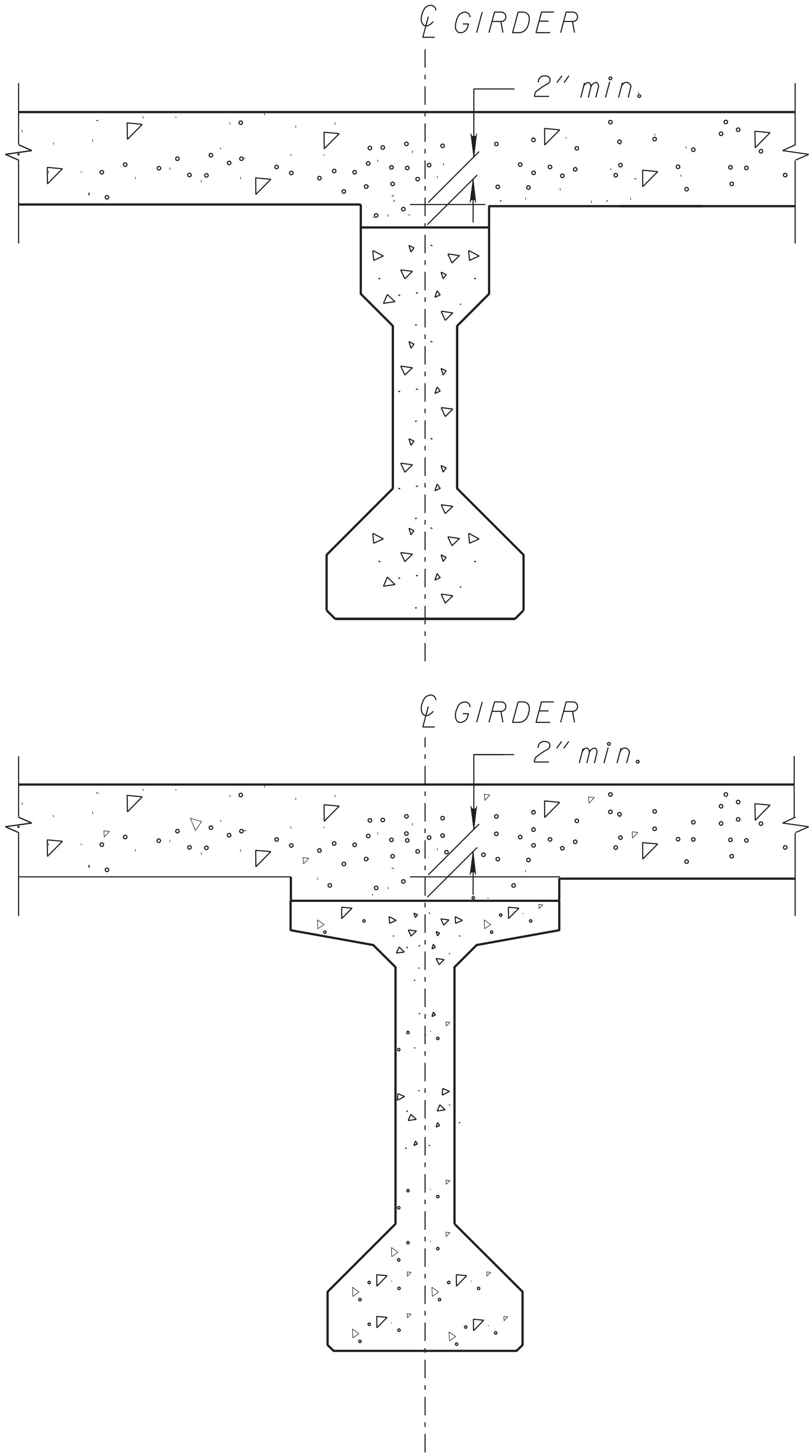


TYPICAL CONCRETE
DECK HAUNCH DETAIL

Figure 302.2.5-1



TYPICAL CONCRETE DECK HAUNCH DETAIL

Figure 302.2.5-2

describes how the quantity of deck concrete was calculated.

[702.7-1] DECK SLAB CONCRETE QUANTITY: The estimated quantity of deck slab concrete is based on the constant deck slab thickness, as shown, plus the quantity of concrete that forms each beam/girder haunch. The estimate assumes a constant haunch thickness of ___ inches and a haunch width equal to the top flange width. Deviate from this haunch thickness as necessary to place the deck surface at the finished grade.

The haunch thickness was measured at the centerline of the beam/girder, from the surface of the deck to the bottom of the top flange minus the deck slab thickness. The area of all embedded steel plates has been deducted from the haunch quantity in accordance with 511.23.

NOTE TO DESIGNER: The note above applies to both rolled beams and plate girders, and measures the haunch to the top of the web (bottom of top flange). The note above applies to new structures with beams/girders placed parallel to the profile grade line. A constant depth haunch may not be practical for new structures whose beams/girders are not placed parallel to the profile grade line. In these special cases, the note shall be modified to fit the exact conditions.

702.8 PRESTRESSED CONCRETE I-BEAM BRIDGES

For prestressed concrete I-beam bridges with concrete deck, compute the concrete topping depth over the top of the beams according to BDM Section 302.5.2.3. Provide a longitudinal cross section showing the topping thickness at each centerline of bearing and at each midspan.

In addition to the deck elevation tables required by BDM Section 702.14, provide the following notes in the plans for every beam:

[702.8-1] CAMBER:

Estimated camber at Day 0 (D_0) is _____ inches.

Estimated camber at Day 30 (D_{30}) is _____ inches.

Deflection due to remaining dead load (e.g. concrete deck, crossframes, diaphragms, barriers, utilities, etc.) is _____ inches.

The beam seat elevations assume estimated camber D_{30} with a sacrificial haunch thickness of 2-inches.

NOTE TO DESIGNER: Refer to BDM Section 302.5.2.3 for description of camber values. If the sacrificial haunch depth differs from 2-inches, revise the note accordingly. In accordance with C&MS 511, the Contractor will adjust the bearing seat elevations based on the actual project schedule. To accommodate this adjustment, Designers shall detail vertical reinforcement in the bearing seats with adjustable lap splices such that the minimum lap length coincides with

D_{30}. Do not include deflection due to the weight of FWS.

[702.8-2] DECK SLAB THICKNESS FOR CONCRETE QUANTITY: The estimated quantity of deck concrete is measured according to C&MS 511. In addition to the design slab thickness, the quantity includes a variable haunch thickness that provides an allowance for: vertical grade adjustment, beam camber and additional sacrificial haunch thickness.

NOTE TO DESIGNER: Delete "vertical grade adjustment" from the above note when the project does not include such an adjustment.

Use the following note when the length of the I-beam, measured along the grade, differs from the length, measured horizontally, by more than 3/8" [10mm]:

[702.8-3] NOTE TO FABRICATOR: The dimensions measured along the length of the beam, marked with a _*_, do not contain an allowance for the effect of the longitudinal grade. Include the proper allowance for these dimensions in the shop drawings.

NOTE TO DESIGNER: Indicate the dimensions that require a grade adjustment with an asterisk or some other easily recognizable symbol and include that symbol in the note above.

702.9 PRESTRESSED CONCRETE BOX BEAM BRIDGE

For prestressed composite concrete box beam bridges with concrete deck, compute the concrete topping depth over the top of the beams according to BDM Section 302.5.1.5. Provide a longitudinal cross section showing the topping thickness at each centerline of bearing and at each midspan.

In addition to the deck elevation tables required by BDM Section 702.14, provide the following notes in the plans for every beam:

[702.9-1] CAMBER:

Estimated camber at Day 0 (D_0) is _____ inches.

Estimated camber at Day 30 (D_{30}) is _____ inches.

Deflection due to remaining dead load (e.g. concrete deck, diaphragms, barriers, utilities, etc.) is _____ inches.

The beam seat elevations assume estimated camber D_{30}.

NOTE TO DESIGNER: Refer to BDM Section 302.5.1.5 for description of camber values. In accordance with C&MS 511, the Contractor will adjust the bearing seat elevations based on the actual project schedule. To accommodate this adjustment, Designers shall detail vertical

1004 LRFD SECTION 4 – STRUCTURAL ANALYSIS AND EVALUATION

S4.4 ACCEPTABLE METHODS OF STRUCTURAL ANALYSIS

This Manual identifies various design conditions that require specific methods of analysis. Where analysis methods are dictated by this Manual, the Designer shall provide justification during the staged review process for designs that utilize alternative analysis methods. This justification shall include impacts to project cost and schedule; safety; constructability; etc. The Department reviewer may consult with the Office of Structural Engineering to determine the appropriateness of the justification. The Department is not responsible for engineering costs incurred as a result of unjustified alternative analysis methods. Where analysis methods are not dictated by this Manual, the selection of an appropriate analysis method utilized for the design of new structures is the responsibility of the Designer.

Regardless of the analysis method utilized for design, superstructures are required to be load rated in accordance with BDM Section 900. At the inventory level, the minimum rating factor for the HL-93 loading shall be 1.0.

Listing design software used for structural analysis in the structure general notes is not required.

S4.5.1 GENERAL

Do not include the stiffness contribution of structurally continuous composite railings, curbs elevated medians and barriers in the structural analysis.

S4.6.2.2.1 APPLICATION

Use the following live load distribution factor application guidelines for *Table 4.6.2.2.1-1* and typical ODOT bridge types:

Typical ODOT Bridge Type	Applicable *Table 4.6.2.2.1-1* Cross-section
Steel beam/girder	(a)
Concrete I-beam	(k)
Composite Box beam	(f)
Non-composite box beam	(g)*

* - Use distribution factors that assume beams are connected only enough to prevent relative displacement at the interface. The tie rods specified in Standard Bridge Drawing PSBD-1-93 do not supply sufficient force to ensure units act together.

The 3.0 ft. limit specified for the roadway part of the overhang, d_e, does not apply to the determination of the interior distribution factor for cross-sections (a) and (k).

S4.6.2.5 EFFECTIVE LENGTH FACTOR, K

In the absence of a refined analysis, the following values for G, as defined in *C4.6.2.5*, may be assumed:

A. For spread footings on rock .... G = 1.5

B. For spread footings on soil.. G = 5.0

C. For footings on multiple rows of piles or drilled shafts:

   End Bearing .... G = 1.0

   Friction.. G = 1.5

D. For footings on a single row of drilled shafts or friction piles...................................... G = 1.0

E. For footings on a single row of end bearing piles...............................refined analysis required

For columns supported on a single row of drilled shafts or friction piles, the effective column length shall include the unbraced length above grade and the depth below grade to the point of fixity. Refer to *Article 10.7.3.13.4* to determine the depth to the point of fixity. For drilled shafts socketed into rock, the point of fixity should be no deeper than the top of rock.

The list above assumes that typical spread footings on rock are anchored when the footing is keyed at least 3 in. into rock including unweathered shale.

S4.6.3 REFINED METHODS OF ANALYSIS

Refer to S*4.4* for limitations placed on refined analysis methods.

S4.6.4.3 APPROXIMATE PROCEDURE

The approximate procedure for moment distribution as described in *Appendix B6* is permitted.

Moment redistribution as described in *Article 5.6.3.4* is not permitted.

S4.7.4.1 GENERAL

If the Designer elects to determine the seismic effects based on the stiffness of the structure and ground acceleration data in lieu of using the connection forces defined in *LRFD 3.10.9.2* and BDM Section 301.4.4.1.b, modal analysis shall be performed in accordance with *LRFD 4.7.4.3.2*.

S4.7.4.4 MINIMUM SUPPORT LENGTH REQUIREMENTS

The minimum support length requirements shall be applied according to BDM Section 301.4.4.1.a.

LRFD SECTION 5 – CONCRETE STRUCTURES

S5.4.2.3.3 SHRINKAGE

Designers shall assume the relative humidity to be 70% in the absence of more precise data.

S5.4.3.3 SPECIAL APPLICATIONS

BDM Section 301.5 and C&MS 509.02 specify all reinforcing steel for structures to be epoxy coated.

S5.4.4.2 MODULUS OF ELASTICITY

Designers shall assume the modulus of elasticity for prestressing strand to be 28,500 ksi in the absence of more precise data.

S5.5.3.1 GENERAL

Fatigue need not be investigated for the design of longitudinal edge beams of slab bridges.

S5.6.3.4 MOMENT REDISTRIBUTION

Moment redistribution is not permitted.

S5.6.4.6 SPIRALS AND TIES

The ratio of spiral reinforcement to total volume of the concrete core (ρ_S) shall be taken as:

$$\rho_S = \frac{4A_S\sqrt{p^2 + (\pi d_S)^2}}{\pi p {d_C}^2}$$

Where:

A_S = Area of spiral reinforcement (in^2)
p = Spiral pitch (in)
d_S = Centerline diameter of the spiral reinforcement = d_C – Bar dia. (in)
d_C = Diameter of the concrete core measured out-to-out of spiral reinforcement (in)

S5.6.7 CONTROL OF CRACKING BY DISTRIBUTION OF REINFORCEMENT

Unless otherwise noted below, the exposure factor (γ_e) for reinforcing steel in cast-in-place concrete bridge decks and cast-in-place slab bridges shall be 0.75. The one inch monolithic wearing surface, BDM Section 302.1.3.1, shall be deducted from both d_c and h.

S5.7.4.4 COHESION AND FRICTION

The top surface of composite prestressed concrete beams produced under Item 515 is intentionally roughened to an amplitude of 0.25 in.

S5.9.2.3.2b TENSION STRESSES

Designers shall assume a severe corrosive environment to determine the tensile stress limit for components with bonded prestressing tendons in non-segmentally constructed bridges.

S5.9.3.4 REFINED ESTIMATES OF TIME-DEPENDENT LOSSES

The refined estimates for time-dependent losses presented in this article may be used for detail design of prestressed members without post-tensioning.

In the absence of more precise data, for prestressed members without post-tensioning, designers may assume the following ages:

A. Age at transfer (t_i) .... 0.75 days
B. Age at deck placement (t_d) .... 45 days
C. Final age (t_f) .... 10,000 days

S5.9.4.1 PRETENSIONING STRAND

The minimum spacing of pretensioning strand shall be 2.0 in. measured center-to-center of the strands.

S5.9.4.3.3 PARTIALLY DEBONDED STRANDS

Refer to BDM Section 302.5.2.2.d for additional debonded strand requirements.

S5.10.1 CONCRETE COVER

The minimum concrete cover for reinforcing steel shall be provided according to BDM Section 301.5.7. No modification for W/C ratio shall be made.

S5.10.2.2 SEISMIC HOOKS

Transverse reinforcement in plastic hinge zones of structures assigned to areas with $0.10 \leq S_{D1} \leq 0.15$ shall be detailed in accordance with *LRFD 5.11.4.1.4*. Plastic hinge zones shall be assumed to extend from the top and bottom of the column a distance taken as the greater of:

A. The maximum cross-sectional dimension of the column,

B. One-sixth of the clear height of the column, or

C. 18.0-in.

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S*5.10.3.1.1 CAST-IN-PLACE CONCRETE*

The maximum nominal aggregate size permitted for structural concrete according to C&MS 499 is 1-inch. When a maximum nominal aggregate size required for design purposes differs from C&MS 499, the Designer shall provide a plan note and specify the concrete pay item "As Per Plan".

S*5.10.3.1.2 PRECAST CONCRETE*

For prestressed concrete mixes, C&MS 515 allows the use of the following aggregate gradations: No. 57, 6, 67, 68, 7, 78 or 8. Unless more precise data is provided, assume the maximum nominal aggregate size according to the No. 57 gradation shown in *C&MS Table 703.01*.

When a maximum nominal aggregate size required for design purposes differs from the gradations specified in C&MS 515, the Designer shall provide a plan note and specify the prestressed concrete pay item "As Per Plan".

S*5.10.4.3 TIES*

Ties are required for T-type and wall-type piers. Refer to BDM Section 303.3.2.8 for more information.

S*5.12.4* *DIAPHRAGMS*

Refer to BDM Section 302.5.2.6 for additional information.

S*5.12.9.5.2* *REINFORCING STEEL*

For 12.0 in., 14.0 in. and 16.0 in. diameter cast-in-place piles, the minimum wall thickness requirements of C&MS 507.06 provide sufficient longitudinal reinforcement to meet *Article 5.12.9.5.2*. Except as noted in BDM Sections 202.2.3.2.b and 303.3.2.5 for capped pile piers, no additional reinforcement is required. The additional steel required for capped pile piers shall extend from the pier cap to a minimum of 15 ft. below the finished ground line or flow line, but is not required to extend 10.0 ft. below the plane where the soil provides adequate lateral restraint.

The cast-in-place concrete piling clear distance requirements specified in *Article 5.12.9.5.2* do not apply to drilled shafts or piles for Capped Pile Piers. Refer to BDM Section 303.4.3 for reinforcing steel requirements in drilled shafts.