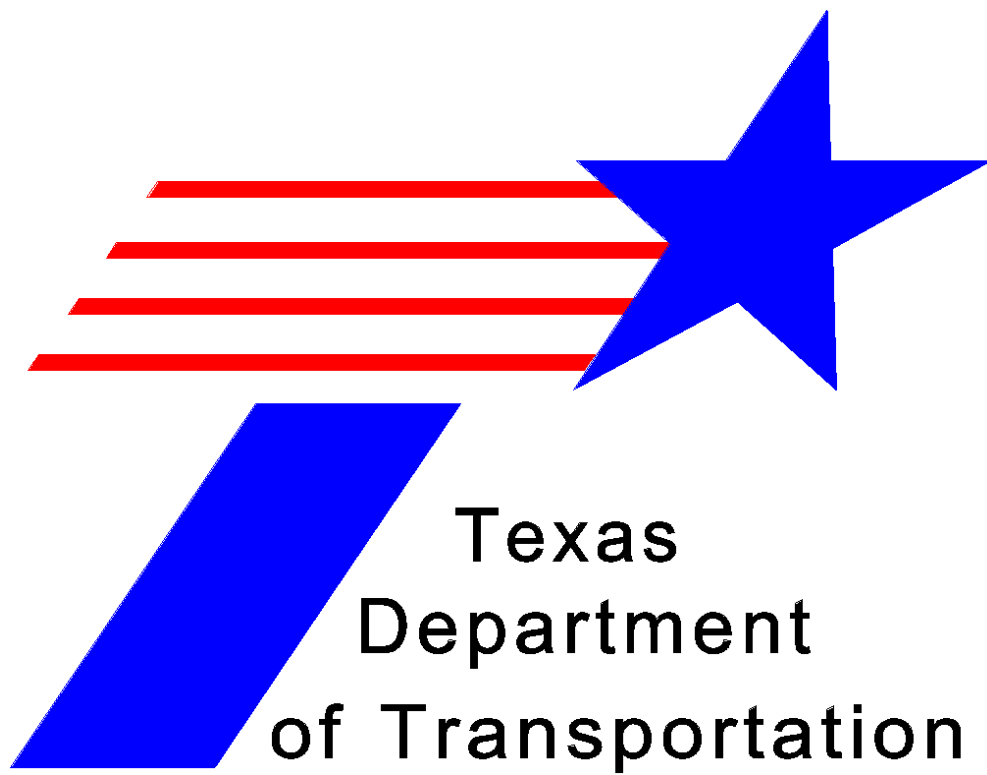


Bridge Design Manual - LRFD



Texas
Department
of Transportation

Revised May 2009

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Manual Notice 2009-1

From: David P. Hohmann, P.E.
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Purpose

This manual documents policy on bridge design in Texas. It assists Texas bridge designers in applying provisions documented in the *AASHTO LRFD Bridge Design Specifications*, to which designers should adhere unless directed otherwise by this document.

Changes

This manual revises policy on foundation load calculations; revises policy on live load deflection check; revises policy on use of empirical design method for slab design; updates prestressed concrete design criteria; revises debonding limits for prestressed concrete design; adds requirements for considering corrosion protection measures; clarifies policy on bearing pad design; revises policy on calculating live load distribution factors for double-tee beams; and corrects minor editorial errors.

This manual has been revised to be current with the *AASHTO LRFD Bridge Design Specifications*, 4th Edition (2007) and 2008 Interim Revisions.

This revision supersedes version 2008-1.

Contact

For more information about any portion of this manual, please contact the Design Section of the Bridge Division.

Archives

Past manual notices are available in a [PDF archive](#).

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Chapter 1

About this Manual

Contents:

Section 1 — Introduction

Section 1

Introduction

Implementation

Load and Resistance Factor Design (LRFD) is a design methodology that makes use of load and resistance factors based on the known variability of applied loads and material properties. In 1994, the American Association of State Highway and Transportation Officials (AASHTO) published the first *AASHTO Load and Resistance Factor Bridge Design Specifications*. The Federal Highway Administration (FHWA) has mandated the use of LRFD for all bridges for which the Texas Department of Transportation (TxDOT) initiates preliminary engineering after October 2007.

Purpose

This manual documents policy on bridge design in Texas. It assists Texas bridge designers in applying provisions documented in the *AASHTO LRFD Bridge Design Specifications*, 4th edition, which designers should adhere to unless directed otherwise by this document. Recommendations and examples are available on the TxDOT web site at http://www.txdot.gov/business/contractors_consultants/bridge/default.htm.

Updates

Updates to this manual are summarized in the following table.

Manual Revision History

Version	Publication Date	Summary of Changes
2005-1	July 2005	New manual
2005-2	September 2005	Revision adding information on deck slabs on slab beams, double-tee beams, and box beams. Also added information on prestressed slab beams, prestressed concrete double-tee beams, prestressed concrete box beams, and cast-in-place concrete slab and girder spans (pan forms).
2006-1	June 2006	Revision adding information on cast-in-place concrete spans, straight plate girders, and curved plate girders.
2006-2	July 2006	Revision adding information in Chapter 3 on prestressed concrete U beams (types U40 and U54) and on concrete deck slabs on U beams (types U40 and U54), and making minor adjustments to references in the Chapter 3 section on geometric constraints for steel-reinforced elastomeric bearings and to the Chapter 4 sections on design criteria for abutments and design criteria for inverted tee reinforced concrete bent caps.

Manual Revision History

Version	Publication Date	Summary of Changes
2007-1	April 2007	Revisions to the design criteria for concrete deck slabs on stringers, cast-in-place concrete slab spans, rectangular reinforced concrete bent caps and inverted tee reinforced concrete bent caps. Revisions to the structural analysis of prestressed concrete I beams, U beams, slab beams and box beams (Types B20, B28, B34 and B40). Revisions to the geometric constraints for abutment and rectangular reinforced concrete bent caps. Revisions to the design recommendations for widenings.
2008-1	April 2008	Revisions to manual to conform with the 4th Edition of the <i>AASHTO LRFD Bridge Design Specifications</i> . Adds information in Chapter 5 for strut-and-tie design method. Revision to requirements for foundation load calculations. Revision to interface shear transfer requirements for prestressed beams. Clarifies policy on live load distribution factors for beam design. Revision to policy on debonding design. Corrects inverted tee design formulas.
2009-1	May 2009	Revisions to the manual to revise policy on foundation load calculations; revises policy on live load deflection check; revises policy on use of empirical design method for slab design; updates prestressed concrete design criteria; revises debonding limits for prestressed concrete design; adds requirements for considering corrosion protection measures; clarifies policy on bearing pad design; revises policy on calculating live load distribution factors for double-tee beams; and corrects minor editorial errors.

Organization

- ◆ The information in this manual is organized as follows:
- ◆ Chapter 1, About this Manual. Introductory information on the purpose and organization of the manual.
- ◆ Chapter 2, Limit States and Loads. General information on limit states and on load factors.
- ◆ Chapter 3, Superstructure Design. Policy on LRFD design of specific bridge superstructure components.
- ◆ Chapter 4, Substructure Design. Policy on LRFD design of specific bridge substructure components.
- ◆ Chapter 5, Other Designs. Design guidelines for bridge widenings and steel-reinforced elastic bearings for prestressed concrete beams and strut-and-tie method. This chapter also addresses corrosion protection measures.

Feedback

Direct any questions or comments on the content of the manual to the Director of the Bridge Division, Texas Department of Transportation.

Chapter 2

Limit States and Loads

Contents:

Section 1 — Limit States

Section 2 — Loads

Section 1

Limit States

Limit States

Classify all bridge designs as typical bridges when applying the operational importance factor, η_I , to strength limit states. Use $\eta_I = 1.0$ for all limit states. See *AASHTO LRFD Bridge Design Specifications*, Article 1.3.5.

Provisions under Extreme Event I need not be considered except for regions near Big Bend.

Provisions under Extreme Event II must be considered only when vehicular collision or vessel collision evaluation is required.

For typical multi-column bridges, determine design loads for foundations at Service Limit State I. Foundation loads for single column bents and other non-typical substructures should be determined by Service Limit State I and Service Limit State IV. For Service Limit State IV, include the vertical wind pressure as specified in Article 3.8.2. For foundation loads on typical multi-column bents and abutments, use the multiple presence factor, m , per *AASHTO LRFD Bridge Design Specifications*, Article 3.6.1.1.2. Distribute the live load equally to all supporting foundations, assuming all lanes are loaded.

Follow the procedures described in the TxDOT Geotechnical Manual, which is available on the internet at <ftp://ftp.dot.state.tx.us/pub/txdot-info/gsd/manuals/geo.pdf>, to determine the size and length of foundations.

Check live load deflection using *AASHTO LRFD Bridge Design Specifications*, Articles 2.5.2.6.2 and 3.6.1.3.2. Ensure that the calculated deflection does not exceed $\text{Span}/800$ using a live load distribution factor equal to number of lanes divided by number of girders. If the bridge has pedestrian sidewalks, the deflection limit is $\text{Span}/1000$.

Section 2

Loads

Live Loads

Use HL93 design live load as described in Article 3.6.1.2 of the *AASHTO LRFD Bridge Design Specifications* unless design for a special vehicle is specified or warranted.

Design widenings for existing structures using HL93. Rate existing structures using *AASHTO Standard Specifications* and HS20 loading. Show load rating and design loads on the bridge plan, for example, HS20 (Existing) HL93 (New).

Do not use the reduction in the multiple presence factor (m) based on Average Daily Truck Traffic (ADTT) on the bridge as suggested in the *AASHTO LRFD Bridge Design Specifications* under commentary on Article 3.6.1.1.2, Multiple Presence of Live Load.

For simple-span bridges, do not apply the provisions for two design trucks as described in Article 3.6.1.3.1 of the *AASHTO LRFD Bridge Design Specifications*.

Disregard recommendations to investigate negative moment and reactions at interior supports for pairs of the design tandem provided in the commentary provided in the *AASHTO LRFD Bridge Design Specifications* under Article 3.6.1.3.1, Application of Design Vehicular Live Loads.

Braking Force

Take the braking force, BR , as 5% of the design truck plus lane load or 5% of the design tandem plus lane load. See Article 3.6.4 in the *AASHTO LRFD Bridge Design Specifications*.

Vehicular Collision Force

See Article 3.6.5 in the *AASHTO LRFD Bridge Design Specifications*. Ongoing TxDOT-sponsored research supports the following policy:

Abutments and retaining walls—Due to the soil behind abutments and retaining walls, the collision force in Article 3.6.5 need not be considered.

Bents—Bents adjacent to roadways with design speeds of 50 mph or less need not meet the requirements of Article 3.6.5. Bents adjacent to roadways with design speeds greater than 50 mph and located within 30 feet of the edge of roadway (defined as edge of lane nearest the column) must meet at least one of the following requirements:

Protect with an approved barrier. The Bridge Division can provide details of acceptable barriers.

Design for 400-kip load. Use a high collision strut between the columns if necessary

Validate that the structure will not collapse by analyzing the structure considering removal of any single column. Analyze using Extreme Event II Limit State. Use 1.25 load factor for all dead loads and 0.5 load factor for live load. Consider live load only on the permanent travel lanes, not the shoulder lanes.

Special considerations include the following:

- ◆ Single-column bents—Generally single-column bents have sufficient mass and will meet the requirements of Article 3.6.5. No further analysis is required for columns with a gross cross-sectional area in excess of 20 sq. ft. and a least dimension of no less than 3 ft.

For structures within 50 ft. of the center line of a railway track, meet the requirements of AREMA or the governing railroad company.

Earthquake Effects

Except for regions near Big Bend that are susceptible to seismic ground motion, disregard provisions such as Article 3.10 in the *AASHTO LRFD Bridge Design Specifications* pertaining to earthquake loads unless specified otherwise.

Vessel Collision

TxDOT requires that all bridges crossing waterways with documented commercial vessel traffic comply with *AASHTO LRFD Bridge Design Specifications*, Article 3.14. For widening of existing structures, at least maintain the current strength of the structure relative to possible vessel impact, and increase the resistance of the structure where indicated if possible. Consult the TxDOT Bridge Division for assistance interpreting and applying these design requirements.

Chapter 3

Superstructure Design

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- Section 1 — Overview
- Section 2 — Concrete Deck Slabs on Stringers
- Section 3 — Concrete Deck Slabs on U Beams (U40 and U54)
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- Section 11 — Cast-in-Place Concrete Slab Spans
- Section 12 — Straight Plate Girders
- Section 13 — Curved Plate Girders

Section 1

Overview

Introduction

This chapter documents policy on Load and Resistance Factor Design (LRFD) of specific bridge superstructure components.

Section 2

Concrete Deck Slabs on Stringers

Materials

Use Class S concrete ($f'_c = 4.0$ ksi). If the deck will be subjected regularly to deicing chemicals based on district policy, add the following plan note: For Class S concrete in slab, use one of the mix design options 1 through 5 required by Item 421.

Use Grade 60 reinforcing steel. Use uncoated reinforcing steel unless the deck will be subjected regularly to de-icing chemicals based on district policy, in which case use epoxy-coated reinforcing steel.

Waterproof deck slabs with one of the two classes of treatment specified in Item 428, “Concrete Surface Treatment,” of the Texas Standard Specifications.

Geometric Constraints

TxDOT standard deck slabs are 8 in. and 8.5 in. with top clear cover of 2 in. and 2.5 in. respectively. Use the 8.5-in deck slab for bridges where regular use of deicing salts is anticipated. Both deck slabs use 1.25-in. bottom clear cover.

Reinforce standard deck slabs as shown in Chapter 5 of the *Bridge Detailing Manual*, available at <http://crossroads.org/gsd/books/det/index.htm>.

Design Criteria

With standard deck slabs, use the following beam spacing and overhang width limits:

- ◆ Maximum clear span (or distance between flange quarter points of steel beams) is 8.667 ft. If permitting use of prestressed concrete panels, ensure clear span is acceptable based on limits shown on standard drawing PCP.
- ◆ Typical overhang is 3.0 ft. measured from the center line of the beam to the edge of the slab.
- ◆ Maximum overhang measured from edge of slab to face of beam top flange (or steel beam flange quarter point) is the lesser of 3.917 ft. or 1.3 times the depth of beam, which prevents excessive torsion on fascia beams during slab placement. At span ends, reduce the limit from 3.917 ft. to 3.083 ft. to account for reduced wheel load distribution.
- ◆ Minimum overhang is 0.5 ft. measured from edge of slab to face of beam top flange to allow sufficient room for the slab drip bead.

Do not use the empirical design method specified in Article 9.7.2 of the *AASHTO LRFD Bridge Design Specifications*. Use the Traditional Design method specified in Article 9.7.3 of the *AASHTO LRFD Bridge Design Specifications*.

Place main reinforcing steel parallel to the skew up to 15 degree skews. Place reinforcing steel perpendicular to beams for skews more than 15 degrees, and use corner breaks. Provide at least #5's at 6 inches for main reinforcement.

Overhang strength for extreme events, described in Article 9.5.5 of the *AASHTO LRFD Bridge Design Specifications*, is satisfied through TxDOT's rail crash testing.

When calculating the cracking moment of a member in accordance with Article 5.7.3.3.2, take the modulus of rupture, f_r , as $0.24 \sqrt{f'_c}$, for all normal weight concrete.

Detailing

In overhangs, use a 3-in. space between outermost Bars D to minimize slab damage from rail impacts. Space Bars T at 9 in. to provide better crack control at construction joints, precast panel ends, and control joints placed at interior bent locations with continuous slab/simple span construction.

The deck slab must be at least 8 inches thick or 8.5 inches thick when deicing chemicals are used.

Section 3

Concrete Deck Slabs on U Beams (U40 and U54)

Materials

Use Class S concrete ($f'_c = 4.0$ ksi). If the deck will be subjected regularly to deicing chemicals based on district policy, add the following plan note: For Class S concrete in slab, use one of the mix design options 1 through 5 required by Item 421.

Use Grade 60 reinforcing steel. Use uncoated reinforcing steel unless the deck will be subjected regularly to de-icing chemicals based on district policy, in which case use epoxy-coated reinforcing steel.

Waterproof deck slabs with one of the two classes of treatment specified in Item 428, “Concrete Surface Treatment,” of the TxDOT Standard Specifications.

Geometric Constraints

TxDOT standard deck slabs are 8 in. and 8.5 in. with top clear cover of 2 in. and 2.5 in. respectively. Use the 8.5-in deck slab for bridges where regular use of deicing salts is anticipated. Both deck slabs use 1.25-in. bottom clear cover.

Reinforce the cast-in-place portion of the slab with #5 bars spaced at 6 in. in the transverse direction and #4 bars spaced at 9 in. in the longitudinal direction

Design Criteria

With standard deck slabs, use the following beam spacing and overhang width limits:

- ◆ Maximum clear span is 8.583 ft. If permitting use of prestressed concrete panels, ensure clear span is acceptable based on limits shown on standard drawing PCP.
- ◆ Typical overhang is 6 ft. 9 in. measured from the center line of the bottom of the exterior beam to the edge of the slab.
- ◆ Maximum overhang is 3.917 ft. measured from slab edge to beam flange edge. At span ends, reduce the 3.917-ft. limit to 3.083 ft. to account for reduced wheel load distribution.
- ◆ Minimum overhang is 0.5 ft. measured from slab edge to beam top flange edge to allow sufficient room for the slab drip bead.
- ◆ For overhangs (measured from the centerline of the outside beam to the edge of the slab) in excess of 7 ft. 3 in., check the outside web-to-bottom flange joint of the exterior beam for adequacy under construction loads.

Do not use the empirical design method specified in Article 9.7.2 of the *AASHTO LRFD Bridge Design Specifications*. Use the Traditional Design method specified in Article 9.7.3 of the *AASHTO LRFD Bridge Design Specifications*.

Place main reinforcing steel parallel to the skew up to 15 degree skews. Place reinforcing steel perpendicular to beams for skews more than 15 degrees, and use corner breaks.

Overhang strength for extreme events, described in Article 9.5.5 of the *AASHTO LRFD Bridge Design Specifications*, is satisfied through TxDOT's rail crash testing.

Detailing

In overhangs, use a 3-in. space between outermost Bars D to minimize slab damage from rail impacts. Space Bars T at 9 in. to provide better crack control at construction joints, precast panel ends, and control joints placed at interior bent locations with continuous slab/simple span construction.

Section 4

Concrete Deck Slabs on Slab Beams, Double-Tee Beams, and Box Beams

Materials

Use Class S concrete ($f'_c = 4.0$ ksi). If the slab will be subjected regularly to deicing chemicals based on district policy, add the following plan note: For Class S concrete in slab, use one of the mix design options 1 through 5 required by Item 421.

Use Grade 60 reinforcing steel. Use uncoated reinforcing steel unless the slab will be subjected regularly to de-icing chemicals based on district policy, in which case use epoxy-coated reinforcing steel.

Waterproof slabs with one of the two classes of treatment specified in Item 428, “Concrete Surface Treatment.”

Geometric Constraints

TxDOT standard composite concrete slabs are 5 in. thick, minimum.

Use 2-in. top clear cover. If the slab will be subjected regularly to deicing chemicals based on district policy, use 2.5-in. top clear cover. No increase in slab thickness is required.

Design Criteria

For transverse reinforcement, use #5 bars spaced at 6-in. maximum for double-tee and slab beam bridges and #4 bars spaced at 6 in. maximum for box beam bridges.

For longitudinal reinforcement, use #4 bars spaced at 12 in. maximum.

Detailing

Use controlled joints at bent centerlines when the slab is continuous over bents.

Section 5

Prestressed Concrete I Beams and I Girders

Materials

Use Class H concrete with a minimum $f'_{ci} = 4.0$ ksi and $f'_c = 5.0$ ksi.

Design beams for 0.5-in. low-relaxation strands. You may use 0.6-in. low-relaxation strands for Type VI beams or other beams as necessary but should check its availability with fabricators.

Use prestressing strand with a specified tensile strength, f_{pu} , of 270 ksi.

Geometric Constraints

Intermediate diaphragms are not required for structural performance. Do not use intermediate diaphragms unless required for erection stability of beam sizes stretched beyond their normal span limits.

Structural Analysis

Beam designs must meet the following requirements:

- ◆ Distribute the weight of one railing to no more than three beams, applied to the composite cross section.
- ◆ Use section properties given on the standard drawings.
- ◆ Composite section properties may be calculated assuming the beam and slab to have the same modulus of elasticity (for beams with $f'_c < 8.5$ ksi). Do not include haunch concrete placed on top of the beam when determining section properties. Section properties based on final beam and slab modulus of elasticity may also be used.
- ◆ Live load distribution factors must conform to *AASHTO LRFD Bridge Design Specifications*, Article 4.6.2.2.2 for flexural moment and Article 4.6.2.2.3 for shear, except as noted below:
 - For exterior beam design with a slab cantilever length equal to or less than one-half of the adjacent interior beam spacing, use the live load distribution factor for the interior beam. The slab cantilever length is defined as the distance from the center line of the exterior beam to the edge of the slab.
 - For exterior beam design with a slab cantilever length exceeding one-half of the adjacent interior beam spacing, use the lever rule with the multiple presence factor of 1.0 for single lane to determine the live load distribution. The live load used to design the exterior beam must never be less than the live load used to design an interior beam.

- Do not use the special analysis based on conventional approximation for loads on piles per *AASHTO LRFD Bridge Design Specifications*, Article C4.6.2.2.2d, unless the effectiveness of diaphragms on the lateral distribution of truck loads is investigated.
- ◆ For interior as well as exterior girders, do not take the live load distribution factor, for moment or shear, as less than

$$\frac{m \times N_L}{N_b}$$
 Where,
 - m = multiple presence factor per AASHTO LRFD 3.6.1.1.2
 - N_L = number of lanes
 - N_b = number of beams or girders
- ◆ When precast concrete deck panels or stay-in-place metal forms are allowed, design the beam using the basic slab thickness.
- ◆ When calculating the cracking moment of a member in accordance with Article 5.7.3.3.2, take the modulus of rupture f_r , as $0.24 \sqrt{f'_c}$, for all normal weight concrete.

Design Criteria

Standard beam designs must meet the following requirements:

- ◆ Strands should be added and depressed in the order shown on the IBD standard drawing, available at <http://www.txdot.gov/insdtdot/orgchart/cmd/cserve/standard/bridge-e.htm>.
- ◆ Use hold-down points shown on the IBD standard drawing, available at <http://www.txdot.gov/insdtdot/orgchart/cmd/cserve/standard/bridge-e.htm>.
- ◆ Strand stress after seating of chucks is not greater than $0.75 f_{pu}$ for low-relaxation strands.
- ◆ Initial tension in the amount of $0.24 \sqrt{f'_{ci}}$ (ksi) is allowed. Based on TxDOT experience, additional bonded reinforcement is not required.
- ◆ Initial compression in the amount of $0.6 f'_{ci}$ (ksi) is allowed.
- ◆ Final stress at the bottom of beam ends need not be checked except when straight debonded strands are used or when the effect of the transfer length of the prestressing strand is considered in the analysis.
- ◆ Final tension in the amount of $0.19 \sqrt{f'_c}$ (ksi) is allowed.

- ◆ The required final concrete strength (f'_c) is typically based on compressive stresses, which must not exceed the following limits:
 - $0.60 f'_c$ for stresses due to total load plus prestress.
 - $0.45 f'_c$ for stresses due to effective prestress plus permanent (dead) loads.
 - $0.40 f'_c$ for stresses due to live loads plus one-half of the sum of stresses due to prestress and permanent (dead) loads.
- ◆ Tension in the amount of $0.24 \sqrt{f'_{ci}}$ is allowed for checking concrete stresses during deck and diaphragm placement.
- ◆ Use an effective strand stress after release of $0.75 f_{pu} - \Delta f_{pES} - \Delta f_{pR1}$.
- ◆ The end position of depressed strands should be as low as possible so that the position of the strands does not control the release strength. Release strength will occasionally be controlled by end conditions when the depressed strands have been raised to their highest possible position.
- ◆ Do not use the simplified procedure for determining shear resistance as allowed by AASHTO LRFD 5.8.3.4.3. Use the General Procedure as provided by AASHTO LRFD 5.8.3.4.2. Do not use provisions of Appendix B of the AASHTO LRFD Bridge Design Specifications.
- ◆ Calculate required stirrup spacing for #4 Grade 60 bars according to the *AASHTO LRFD Bridge Design Specifications*, Article 5.8. Change stirrup spacing as shown on IBD standard drawing, available at <http://www.txdot.gov/insdtdot/orgchart/cmd/cserve/standard/bridge-e.htm>, only if analysis indicates the inadequacy of the standard design.
- ◆ Replace AASHTO LRFD equation 5.8.4.2-1 with the following:

$$V_{ui} = \frac{V_{u1}Q}{Ib_{vi}}$$

Take b_{vi} , width of the interface, equal to the beam top flange width. Do not reduce b_{vi} to account for precast panel bedding strips.
- ◆ Determine interface shear transfer in accordance with AASHTO LRFD 5.8.4. Take Cohesion and Friction Factors as provided in AASHTO LRFD Article 5.8.4.3 as follows:
 - $c = 0.28$ ksi
 - $\mu = 1.0$
 - $K_1 = 0.3$
 - $K_2 = 1.8$ ksi

- ◆ Replace AASHTO LRFD equation 5.4.2.3.2-2 with the following:

$$k_s = 1.45 - 0.13 (V/S) \geq 0.0$$
- ◆ Compute deflections due to slab weight and composite dead loads assuming the beam and slab to have the same modulus of elasticity. Assume $E_c = 5,000,000$ psi for beams with $f'_c < 8.5$ ksi. Predicted slab deflections should be shown on the plans although field experience indicates actual deflections are generally less than predicted. Use the deflection due to slab weight only times 0.8 for calculating haunch depth.
- ◆ TxDOT standard I beams reinforced as shown on the IBD standard drawing are adequate for the *AASHTO LRFD Bridge Design Specifications* requirements of Article 5.10.10.
- ◆ A calculated positive (upward) camber is required after application of all permanent (dead) loads.

When calculating prestress losses, use *AASHTO LRFD Bridge Design Specifications 2004*, 3rd. Ed., Article 5.9.5 Loss of Prestress. In the absence of *AASHTO LRFD Bridge Design Specifications 2004*, use *AASHTO LRFD Bridge Design Specifications 2007*, 4th Ed., Article 5.9.5 Loss of Prestress with the following changes.

- ◆ Replace AASHTO LRFD (2007) Eq. 5.9.5.1-1 with the following:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$$

- ◆ Replace AASHTO LRFD (2007) Eq. 5.9.5.1-2 with the following:

$$\Delta f_{pT} = \Delta f_{pF} + \Delta f_{pA} + \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$$

Where,

$$\Delta f_{pT} = \text{total loss (ksi)}$$

$$\Delta f_{pF} = \text{loss due to friction (ksi)}$$

$$\Delta f_{pA} = \text{loss due to anchorage set (ksi)}$$

$$\Delta f_{pES} = \text{loss due to elastic shortening (ksi)}$$

$$\Delta f_{pSR} = \text{loss due to shrinkage (ksi)}$$

Δf_{pCR} = loss due to creep of concrete (ksi)

Δf_{pR2} = loss due to relaxation of steel after transfer (ksi)

- ◆ Add the following to AASHTO LRFD (2007) Article 5.9.5.1:

"In pretensioned members, where the approximate lump sum estimate of losses specified in

Article 5.9.5.3 is used, the part of the loss due to relaxation occurring before transfer, Δf_{pR1} , should be deducted from the total relaxation.

For post-tensioned members, consideration should be given to a loss of tendon force, as indicated by pressure readings, within the stressing equipment."

- ◆ In the AASHTO LRFD (2007) Eq. 5.9.5.2.3a-1, replace E_{ct} with E_{ci} where E_{ci} = modulus of elasticity of concrete at transfer (ksi).

- ◆ Add the following to AASHTO LRFD (2007) Article 5.9.5.2.3a:

"For pretensioned components of usual design, f_{cgp} may be calculated on the basis of a prestressing steel stress assumed to be $0.65 f_{pu}$ for stress-relieved strand and high-strength bars and $0.70 f_{pu}$ for low relaxation strand.

For components of unusual design, more accurate methods supported by research or experience should be used."

- ◆ Disregard all commentary in AASHTO LRFD (2007) Article C5.9.5.2.3a until Equation C5.9.5.2.3a-1.

- ◆ Disregard AASHTO LRFD (2007) Equation 5.9.5.3-1.

For low relaxation strands, the values for I-beams and I-girders in AASHTO LRFD Table 5.9.5.3-1 may be reduced by 6.0 ksi.

- ◆ Add the following to AASHTO LRFD (2007) Table 5.9.5.3-1:

Type of Beam Section	Level	For Wires and Strands with $f_{pu} = 235, 250 \text{ or } 270 \text{ ksi}$	For Bars with $f_{pu} = 145 \text{ or } 160 \text{ ksi}$
I-Girder	Average	$33.0 \left[1.0 - 0.15 \frac{f'_c - 6.0}{6.0} \right] + 6.0PPR$	$19.0 + 6.0PPR$

- ◆ Replace AASHTO LRFD (2007) Article 5.9.5.4 with the following:

5.9.5.4 Refined Estimates of Time-Dependent Losses

5.9.5.4.1 General

More accurate values of creep-, shrinkage-, and relaxation-related losses, than those specified in Article 5.9.5.3 may be determined in accordance with the provisions of this article for prestressed members with:

- Spans not greater than 250 ft.,
- Normal weight concrete, and
- Strength in excess of 3.50 ksi at the time of prestress.

For lightweight concrete, loss of prestress shall be based on the representative properties of the concrete to be used.

For segmental construction, for all considerations other than preliminary design, prestress losses shall be determined as specified in Article 5.9.5, including consideration of the time-dependent construction method and schedule shown in the contract documents.

5.9.5.4.2 Shrinkage

Loss of prestress, in ksi, due to shrinkage may be taken as:

- For pretensioned members:

$$\Delta f_{pSR} = (17.0 - 0.150H) \quad (5.9.5.4.2-1)$$

- For post-tensioned members:

$$\Delta f_{pSR} = (13.5 - 0.123H) \quad (5.9.5.4.2-2)$$

Where,

H = the average annual ambient relative humidity (percent)

5.9.5.4.3 Creep

Prestress loss due to creep may be taken as:

$$\Delta f_{pCR} = 12.0 f_{cgp} - 7.0 \Delta f_{cdp} \geq 0 \quad (5.9.5.4.3-1)$$

Where,

f_{cgp} = concrete stress at center of gravity of prestressing steel at transfer (ksi)

Δf_{cdp} = change in concrete stress at center of gravity of prestressing steel due to permanent loads, with the exception of the load acting at the time the prestressing force is applied.

Values of Δf_{cdp} should be calculated at the same section or at sections for which f_{cgp} is calculated (ksi).

5.9.5.4.4 Relaxation

5.9.5.4.4a General

The total relaxation at any time after transfer shall be taken as the sum of the losses specified in Articles 5.9.5.4.4b and 5.9.5.4.4c.

5.9.5.4.4b At Transfer

In pretensioned members, the relaxation loss in prestressing steel, initially stressed in excess of $0.50f_{pu}$, may be taken as:

- For stress-relieved strand:

$$\Delta f_{pR1} = \frac{\log(24.0t)}{10.0} \left[\frac{f_{pj}}{f_{py}} - 0.55 \right] f_{pj}$$

(5.9.5.4.4b-1)

- For low-relaxation strand:

$$\Delta f_{pR1} = \frac{\log(24.0t)}{40.0} \left[\frac{f_{pj}}{f_{py}} - 0.55 \right] f_{pj}$$

(5.9.5.4.4b-2)

Where,

t = time estimated in days from stressing to transfer (day)

f_{pj} = initial stress in the tendon at the end of stressing (ksi)

f_{py} = specified yield strength of prestressing steel (ksi)

5.9.5.4.4c After Transfer

Losses due to relaxation of prestressing steel may be taken as:

- For pretensioning with stress-relieved strands:

$$\Delta f_{pR2} = 20.0 - 0.4\Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})$$

(5.9.5.4.4c-1)

- For post-tensioning with stress-relieved strands:

$$\Delta f_{pR2} = 20.0 - 0.3\Delta f_{pF} - 0.4\Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})$$

(5.9.5.4.4c-2)

Where,

Δf_{pF} = the friction loss below the level of $0.70f_{pu}$ at the point under consideration, computed according to Article 5.9.5.2.2 (ksi)

Δf_{pES} = loss due to elastic shortening (ksi)

Δf_{pSR} = loss due to shrinkage (ksi)

Δf_{pCR} = loss due to creep of concrete (ksi)

- For prestressing steels with low relaxation properties conforming to AASHTO M 203 (ASTM A416 or E328):

Use 30 percent of Δf_{pR2} given by Eq. 1 or 2.

- For post-tensioning with 145 to 160 ksi bars:
Loss due to relaxation should be based on approved test data. If test data is not available, the loss may be assumed to be 3.0 ksi.

Section 6

Prestressed Concrete U Beams (Types U40 and U54)

Materials

Use Class H concrete with a minimum $f'_{ci} = 4.0$ ksi and $f'_c = 5.0$ ksi.

Design beams for 0.5-in. low-relaxation strands. You may use 0.6-in. low-relaxation strands for unusual cases but should check its availability with fabricators.

Use prestressing strand with a specified tensile strength f_{pw} of 270 ksi.

You need not increase section properties of the beam to account for the transformed area of strands or mild steel.

Geometric Constraints

The maximum skew angle for U-beam bridges is 45 degrees.

Structural Analysis

Beam designs must meet the following requirements:

- ◆ Include the overlay at the discretion of the designer or if the bridge will receive the overlay immediately after construction. Recognize that including the overlay in the design of U beams can significantly limit their ability to span longer span lengths.
- ◆ Distribute 2/3 of the rail dead load to the exterior beam and 1/3 of the rail dead load to the adjacent interior beam applied to the composite cross section.
- ◆ Each U beam has two interior diaphragms at a maximum average thickness of 13 in. They are located as close as 10 ft. from midspan of the beam. Account for each diaphragm as a 2-kip load for U40 beams and as a 3-kip load for U54 beams applied to the non-composite cross section.
- ◆ Use section properties given on the standard drawings.
- ◆ Composite section properties may be calculated assuming the beam and slab to have the same modulus of elasticity (for beams with $f'_c < 8.5$ ksi). Do not include haunch concrete placed on top of the beam when determining section properties. Section properties based on final beam and slab modulus of elasticity may also be used.
- ◆ Live load distribution factors for interior beams must conform to *AASHTO LRFD Bridge Design Specifications*, Article 4.6.2.2.2 for flexural moment and Article 4.6.2.2.3 for shear.

- ◆ Live load distribution factors for exterior beams must conform to *AASHTO LRFD Bridge Design Specifications*, Article 4.6.2.2.2 for flexural moment and Article 4.6.2.2.3 for shear, with the following exceptions:
 - When using the lever rule, multiply the result of the lever rule by 0.9 to account for continuity.
 - When the clear roadway width is **greater** than or equal to 20.0 ft, use a distribution factor for two or more design lanes loaded only. Do not design for one lane loaded.
 - When the clear roadway width is less than 20.0 ft, design for one lane loaded with a multiple presence factor of 1.0.

The live load used to design the exterior beam must never be less than the live load used to design an interior beam.

- ◆ For bridges with less than three girders in the cross section, assume the live load distribution factors for flexural moment and shear are equal to the number of lanes divided by the number of girders. Determine the number of lanes as required by *AASHTO LRFD Bridge Design Specifications*, Article 3.6.1.1.1.
- ◆ For interior as well as exterior girders, do not take the live load distribution factor, for moment or shear, as less than

$$\frac{m \times N_L}{N_b}$$

Where,

m = multiple presence factor per AASHTO LRFD 3.6.1.1.2

N_L = number of lanes

N_b = number of beams or girders

Design Criteria

Standard beam designs must meet the following requirements:

- ◆ Stresses at the ends of the beam are controlled with the use of debonding. Draped strands are not permitted in U beams.
- ◆ Debonded strands must conform to AASHTO LRFD Bridge Design Specifications, Article 5.11.43 except as noted below:
 - Debond no more than 75 percent of the total number of strands.

- Debond no more than 75 percent of the number of strands in that row.
 - The maximum debonding length must be the lesser of one-half the span length minus the maximum development length, 0.2 times the beam length, or 15 feet.
 - Not more than 75 percent of the debonded strands, or 10 strands, whichever is greater, shall have the debonding terminated at any section, where section is defined as an increment (e.g. 3 feet, 6 feet, 9 feet).
- ◆ Grouping of U-beam designs are at the discretion of the designer. However, no exterior U beam may have less carrying capacity than that of an interior U beam of equal length. If the designer chooses to group beams, a general rule is to group beams with no more than a four-strand difference.
 - ◆ See [Prestressed Concrete I Beams and I Girders](#) for other design criteria.

Detailing

Detail span sheets for a cast-in-place slab with precast concrete panels.

Section 7

Prestressed Concrete Slab Beams

Materials

Use Class H concrete with a minimum f'_{ci} of 4.0 ksi and f'_c of 5.0 ksi.

Design beams for 0.5-in. low-relaxation strands.

Use prestressing strand with a specified tensile strength, f_{pu} , of 270 ksi.

Geometric Constraints

The maximum skew angle for slab beam bridges is 30 degrees.

The minimum gap between adjacent slab beams is 0.5-in. A preferable gap range is 1 in. to 1.5 in.

A 5-in. minimum thickness composite concrete slab is required.

Structural Analysis

Beam designs must meet the following requirements:

- ◆ Distribute the weight of one railing to no more than three beams, applied to the composite cross section.
- ◆ Use section properties given on the prestressed concrete slab beam standard drawings.
- ◆ Composite section properties may be calculated assuming the beam and slab overlay have the same modulus of elasticity (for beams with $f'_c < 8.5$ ksi). Do not include haunch concrete placed on top of the beam when determining section properties. Section properties based on final beam and slab modulus of elasticity may also be used.
- ◆ Live load distribution factors for all beams, both moment and shear, must conform to *AASHTO LRFD Bridge Design Specifications*, Table 4.6.2.2.2b-1, using cross section(g), if the beams are connected only enough to prevent relative vertical displacement at their interfaces. This is called *S/D* distribution.
- ◆ Do not apply the skew correction factors for moment as suggested in Article 4.6.2.2.2e nor for shear as suggested in Article 4.6.2.2.3c.

- ◆ For interior as well as exterior girders, do not take the live load distribution factor, for moment or shear, as less than

$$\frac{m \times N_L}{N_b}$$

Where,

m = multiple presence factor per AASHTO LRFD 3.6.1.1.2

N_L = number of lanes

N_b = number of beams or girders

Design Criteria

Standard beam designs must meet the following requirements:

- ◆ Add strands in the order shown on the PSBNS standard drawing, available at <http://www.dot.state.tx.us/insdot/orgchart/cmd/cserve/standard/bridge-e.htm>.
- ◆ Debond strands in 3-ft. increments at beam ends if necessary to control stresses at release.
- ◆ Debonded strands must conform to AASHTO LRFD Bridge Design Specifications, Article 5.11.43 except as noted below:
 - Debond no more than 75 percent of the total number of strands.
 - Debond no more than 75 percent of the number of strands in that row.
 - The maximum debonding length must be the lesser of one-half the span length minus the maximum development length, 0.2 times the beam length, or 15 feet.
 - Not more than 75 percent of the debonded strands, or 10 strands, whichever is greater, shall have the debonding terminated at any section, where section is defined as an increment (e.g. 3 feet, 6 feet, 9 feet).
- ◆ Calculate required stirrup spacing for #4 Grade 60 bars according to the *AASHTO LRFD Bridge Design Specifications*, Article 5.8. Change stirrup spacing as shown on relevant standard drawings available at <http://www.dot.state.tx.us/insdot/orgchart/cmd/cserve/standard/bridge-e.htm> only if analysis indicates inadequacy of the standard design.
- ◆ TxDOT standard slab beams satisfy Article 5.8.4 and Article 5.10.10 of the *AASHTO LRFD Bridge Design Specifications*.

- ◆ Compute deflections due to slab weight and composite dead loads assuming the beam and slab to have the same modulus of elasticity. Assume $E_c = 5,000,000$ psi for beams with $f'_c < 8.5$ ksi. Show predicted slab deflections on the plans even though field experience indicates actual deflections are generally less than predicted. Use the deflection due to slab weight only times 0.8 for calculating haunch depth.
- ◆ See Section 5, [Prestressed Concrete I Beams and I Girders](#) for other design criteria.

Section 8

Prestressed Concrete Double-Tee Beams

Materials

Use Class H concrete with a minimum f'_{ci} of 4.0 ksi and f'_c of 5.0 ksi.

Use non-shrink cementitious grout for shear keys.

Design beams for 0.5-in. low-relaxation strands.

Use prestressing strand with a specified tensile strength, f_{pu} , of 270 ksi.

Geometric Constraints

Double-tee standard drawings do not accommodate skewed bridges.

A 5-in. minimum thickness composite concrete slab or 2-in. minimum thickness asphaltic concrete pavement (ACP) overlay is required.

Six-ft. wide beams are required for fascia positions when beams are not topped with a 5-in. slab.

Structural Analysis

Beam designs must meet the following requirements:

- ◆ Distribute the weight of one railing to no more than three beams.
- ◆ Use section properties provided on the prestressed concrete double-tee beam standard drawings.
- ◆ Composite section properties may be calculated assuming the beam and composite concrete slab overlay have the same modulus of elasticity (for beams with $f'_c < 8.5$ ksi). When determining section properties, do not include haunch concrete placed on top of the beam. Section properties based on final beam and slab modulus of elasticity may also be used.
- ◆ Regardless of topping, live load distribution factors for all beams, both interior as well as exterior, and both moment and shear, must conform to *AASHTO LRFD Bridge Design Specifications*, Table 4.6.2.2.2b-1, using cross section (i) if beams are connected only enough to prevent relative vertical displacement at their interfaces. Use $K = 2.2$ when determining the live load distribution factor. Use $S/10$ as maximum limit on live load distribution.
- ◆ Do not apply the skew correction factors for moment as suggested in Article 4.6.2.2.2e nor for shear as suggested in Article 4.6.2.2.3c.

- ◆ For interior as well as exterior girders, do not take the live load distribution factor, for moment or shear, as less than

$$\frac{m \times N_L}{N_b}$$

Where,

m = multiple presence factor per AASHTO LRFD 3.6.1.1.2

N_L = number of lanes

N_b = number of beams or girders

Design Criteria

Standard beam designs must meet the following requirements:

- ◆ Strands should be added and depressed in the order shown on the DTBND standard drawings, available at http://www.dot.state.tx.us/insdtdot/orgchart/cmd/cserve/s_tandard/bridge-e.htm.
- ◆ Use hold-down points shown on the DTBS and DTBO standard drawings, available at <http://www.dot.state.tx.us/insdtdot/orgchart/cmd/cserve/standard/bridge-e.htm>.
- ◆ The end position of depressed strands should be as low as possible so that the position of the strands does not control the release strength. Release strength is occasionally controlled by end conditions when the depressed strands have been raised to their highest possible position.
- ◆ Calculate required stirrup spacing for #3 Grade 60 bars according to the *AASHTO LRFD Bridge Design Specifications*, Article 5.8. Change stirrup spacing as shown on the DTBS and DTBO standard drawings, available at <http://www.dot.state.tx.us/insdtdot/orgchart/cmd/cserve/standard/bridge-e.htm>, only if analysis indicates the inadequacy of the standard design.
- ◆ TxDOT standard double-tee beams satisfy Article 5.8.4 of the *AASHTO LRFD Bridge Design Specifications*.
- ◆ For double-tee beams with a composite concrete slab overlay, compute deflections due to slab weight and composite dead loads assuming the beam and slab to have the same modulus of elasticity. Assume $E_c = 5,000,000$ psi for beams with $f'_c < 8.5$ ksi. Show predicted slab deflections on the plans even though field experience indicates that actual deflections are generally less than predicted. Use the deflection due to slab weight only times 0.8 for calculating haunch depth.
- ◆ TxDOT standard double-tee beams satisfy Article 5.10.10 of the *AASHTO LRFD Bridge Design Specifications*.

- ◆ Connect adjacent beams with lateral connectors, shown on standard drawings DTBS and DTBO, spaced at 5 ft. maximum, with the first lateral connectors set 5 ft. from bent centerlines. See span standard drawings for completion of lateral connection details.
- ◆ See Section 5, [Prestressed Concrete I Beams and I Girders](#), for other design criteria.

Section 9

Prestressed Concrete Box Beams (Types B20, B28, B34, and B40)

Materials

Use Class H concrete with a minimum of $f'_{ci} = 4.0$ ksi and $f'_c = 5.0$ ksi.

Use Class S concrete ($f'_c = 4.0$ ksi) for shear keys.

Design beams for 0.5-in low-relaxation strands.

Use prestressing strand with a specified tensile strength, f_{pu} , of 270 ksi.

Geometric Constraints

The maximum skew angle for box beam bridges is 30 degrees.

The maximum gap between adjacent box beams is 2 in. The minimum gap between adjacent box beams is 1 in.

A 5-in. minimum thickness composite concrete slab overlay or 2-in. minimum thickness asphaltic concrete pavement (ACP) overlay is required.

Structural Analysis

Beam designs must meet the following requirements:

- ◆ Distribute the weight of one railing to no more than three beams.
- ◆ Use section properties given on the prestressed concrete box beam standard drawings.
- ◆ Composite section properties may be calculated assuming the beam and composite concrete slab overlay have the same modulus of elasticity (for beams with $f'_c < 8.5$ ksi). When determining section properties, do not include haunch concrete placed on top of the beam. Section properties based on final beam and slab modulus of elasticity may also be used.
- ◆ Live load distribution factors must conform to *AASHTO LRFD Bridge Design Specifications*, Article 4.6.2.2.2 and Article 4.6.2.2.3. Use:
 - Cross section (f) with bridges having a composite concrete slab
 - Cross section (g) with bridges having ACP applied directly to tops of beams, assuming beams are sufficiently connected to act as a unit.
- ◆ Do not apply the skew correction factor for moment as suggested in Article 4.6.2.2.2e.

- ◆ For interior as well as exterior girders, do not take the live load distribution factor, for moment or shear, as less than

$$\frac{m \times N_L}{N_b}$$

Where,

m = multiple presence factor per AASHTO LRFD 3.6.1.1.2

N_L = number of lanes

N_b = number of beams or girders

Design Criteria

Standard beam designs must meet the following requirements:

- ◆ Strands should be added and debonded in the order shown on the BBND standard drawings, available at <http://www.dot.state.tx.us/insdtdot/orgchart/cmd/cserve/standard/bridge-e.htm>.
- ◆ Debond strands in 3-ft. increments at beam ends if necessary to control stresses at release.
- ◆ Debonded strands must conform to AASHTO LRFD Bridge Design Specifications, Article 5.11.43 except as noted below:
 - Debond no more than 75 percent of the total number of strands.
 - Debond no more than 75 percent of the number of strands in that row.
 - The maximum debonding length must be the lesser of one-half the span length minus the maximum development length, 0.2 times the beam length, or 15 feet.
 - Not more than 75 percent of the debonded strands, or 10 strands, whichever is greater, shall have the debonding terminated at any section, where section is defined as an increment (e.g. 3 feet, 6 feet, 9 feet).
- ◆ Calculate required stirrup spacing for #4 Grade 60 bars according to the *AASHTO LRFD Bridge Design Specifications*, Article 5.8. Change stirrup spacing as shown on relevant standard drawings, available at <http://www.dot.state.tx.us/insdtdot/orgchart/cmd/cserve/standard/bridge-e.htm>, only if analysis indicates the inadequacy of the standard design.
- ◆ TxDOT standard box beams satisfy Article 5.8.4 and Article 5.10.10 of the *AASHTO LRFD Bridge Design Specifications*.

-
- ◆ For box beams with a composite concrete slab overlay, compute deflections due to slab weight and composite dead loads assuming the beam and slab to have the same modulus of elasticity. Assume $E_c = 5,000,000$ psi for beams with $f'_c < 8.5$ ksi. Show predicted slab deflections on the plans even though field experience indicates actual deflections are generally less than predicted. Use the deflection due to slab weight only times 0.8 for calculating haunch depth.
 - ◆ Use shear keys for all box beam bridges. Do not consider composite action between beams and shear keys in computing live load distribution factors, nor for strength stress or deflection calculations.
 - ◆ Transverse post-tensioning is required for box beam bridges topped with an ACP overlay applied directly to the tops of beams. Space tendons at 10 ft. maximum with the first tendons set 10 ft. from bent centerlines. Post-tensioning details are provided on standard drawing BBCDO.
 - ◆ See Section 5, [Prestressed Concrete I Beams and I Girders](#) for other design criteria.

Section 10

Cast-in-Place Concrete Slab and Girder Spans (Pan Forms)

Materials

Use Class S concrete ($f'_c = 4.0$ ksi). If the slab will be subjected regularly to deicing chemicals based on district policy, add the following plan note: For Class S concrete in slab, use one of the mix design options 1 through 5 required by Item 421.

Geometric Constraints

The only skew angles available for pan form spans are provided on relevant standard drawings.

The only span lengths available for pan form spans are provided on relevant standard drawings.

Limit slab overhangs to a maximum of 13.75 inches measured from face of stem to edge of slab.

Structural Analysis

None required.

Design Criteria

None required. Pan form spans are predesigned and shown on standard drawings.

Section 11

Cast-in-Place Concrete Slab Spans

Materials

Use Class S concrete ($f'_c = 4.0$ ksi). If the slab will be subjected regularly to deicing chemicals based on district policy, add the following plan note: For Class S concrete in slab, use one of the mix design options 1 through 5 required by Item 421.

Use Grade 60 reinforcing steel. Use uncoated reinforcing steel unless the slab will be subjected regularly to de-icing chemicals based on district policy, in which case use epoxy-coated reinforcing steel.

Waterproof slabs with one of the two classes of treatment specified in Item 428, “Concrete Surface Treatment.”

Geometric Constraints

The maximum skew angle for slab span bridges is 30 degrees. Use shear keys 2 in. deep by 6 in. wide, parallel to traffic, with skewed spans. Shear keys should be formed into the top of substructure caps in the middle of the caps. See [standard drawings](#) for shear key details.

Break slab corners 1.5 ft. with skews more than 15 degrees.

Minimum slab depths from *AASHTO LRFD Bridge Design Specifications*, Table 2.5.2.6.3-1 are guidelines but are not required.

Use a top clear cover of 2 in. Use 2.5-in. top clear cover where regular use of deicing salts is anticipated. Use 1.25-in. bottom clear cover.

Limit span lengths to approximately 25 ft. for simple spans and end spans of continuous units. Limit interior spans of continuous units to approximately 30 ft.

Structural Analysis

Distribute the weight of all railing and sidewalks over the entire slab width if the slab is no wider than 32 ft. Otherwise, distribute railing load over 16 ft.

Design using 1-ft. wide strips. Take bearing centerline at cap quarter points. For interior supports of continuous spans, assume bearing centerline coincides with cap centerline.

Apply both the axle loads and lane loads of the HL-93 live load in accordance with *AASHTO LRFD Bridge Design Specifications*, Article 3.6.1.3.3 for spans more than 15 ft.

Distribute live load in accordance with *AASHTO LRFD Bridge Design Specifications*, Article 4.6.2.3 using Equation 4.6.2.3-2. Use Equation 4.6.2.3-3 to reduce force effects with skewed bridges.

For longitudinal edge beams, required by *AASHTO LRFD Bridge Design Specifications*, Articles 5.14.4.1 and 9.7.1.4, apply one line of wheels plus the tributary portion of the lane load to the reduced strip width specified in Article 4.6.2.1.4b.

Design Criteria

Shear design is not required when spans are designed in accordance with *AASHTO LRFD Bridge Design Specifications*, Article 4.6.2.3.

The longitudinal edge beam cannot have less flexural reinforcement than interior slab regions. Do not consider concrete barrier rails, parapets, or sidewalks in longitudinal edge beam design.

Provide bottom transverse distribution reinforcement. Use *AASHTO LRFD Bridge Design Specifications*, Equation 5.14.4.1-1 to determine the required amount.

Provide #4 reinforcing bars at 12-in. maximum spacing for shrinkage and temperature reinforcement required to satisfy *AASHTO LRFD Bridge Design Specifications*, Article 5.10.8.

Assume Class 1 exposure condition when checking distribution of reinforcement for crack control except for top flexural reinforcement in continuous spans, in which case assume Class 2 exposure condition.

When calculating the cracking moment of a member in accordance with Article 5.7.3.3.2, take the modulus of rupture, f_r , as $0.24 \sqrt{f'_c}$, for all normal weight concrete.

Section 12

Straight Plate Girders

Materials

Use A 709 Grade 50W steel for unpainted bridges. Use A 709 Grade 50 steel for painted bridges. You can use A 709 Grade HPS 70W steel for both unpainted and painted bridges if it is economical or otherwise beneficial to do so.

Use 0.875-in. or 1-in. diameter bolts for bolted connections.

Geometric Constraints

Minimum flange width is $0.25D$, where D =web depth, but not less than 15 in.

Minimum flange thickness is 0.75 in.

Minimum web thickness is 0.50 in.

Minimum stiffener thickness used to connect cross frames or diaphragms to girder is 0.50 in.

Structural Analysis

Beam designs must meet the following requirements:

- ◆ Distribute the weight of one railing to no more than three girders, applied to the composite cross section.
- ◆ Assume no slab haunch when determining composite section properties.
- ◆ Live load distribution factors must conform to *AASHTO LRFD Bridge Design Specifications*, Article 4.6.2.2.2 for flexural moment and Article 4.6.2.2.3 for shear, except as follows:
 - For exterior girder design with a slab cantilever equal to or less than half the adjacent girder interior spacing, use the live load distribution factor for the interior girder. The slab cantilever is the distance from the centerline of the exterior girder to the edge of the slab.
 - For exterior girder design with a slab cantilever length greater than half the adjacent interior girder spacing, use the lever rule with the multiple presence factor of 1.0 for single lane to determine the live load distribution. The live load used to design the exterior girder must never be less than the live load used to design an interior girder.

- ◆ For interior as well as exterior girders, do not take the live load distribution factor, for moment or shear, as less than

$$\frac{m \times N_L}{N_b}$$

Where,

m = multiple presence factor per AASHTO LRFD 3.6.1.1.2

N_L = number of lanes

N_b = number of beams or girders

- ◆ When checking the Fatigue and Fracture Limit State, remove the 1.2 multiple presence factor from the one-design-lane-loaded empirical live load distribution factors.
- ◆ Use only one lane of live load in the structure model when checking the Fatigue and Fracture Limit State.

Design Criteria

Standard beam designs must meet the following requirements:

- ◆ Regarding *AASHTO LRFD Bridge Design Specifications*, Article 6.7.2, do not specify girders to be out-of-plumb in the steel-dead-load-only or theoretical-no-load condition. Diaphragms and cross frames have traditionally been installed with girders plumb and no significant problems have been reported to date. If the designer believes that analysis indicates that girders will be significantly beyond plumb after slab concrete is placed, contact the Director of the Bridge Division for guidance.
- ◆ Diaphragm and cross-frame designs must meet the following requirements:
 - The maximum spacing is 30 ft. if all limit states requirements are met.
 - Provide diaphragms/cross frames at all end bearings. At least two interior bearings at a bent must have a diaphragm/cross frame intersecting them.
 - Set diaphragms/cross frames parallel to skew up to 20 degrees. Set radial to girders beyond 20 degrees.
 - Check the limiting slenderness ratio of cross-frame members using bracing compression member criteria provided in *AASHTO LRFD Bridge Design Specifications*, Article 6.9.3.

Girder designs must meet the following requirements:

- ◆ Use composite design and place shear connectors the full girder length.
- ◆ Do not use longitudinal stiffeners unless web depth exceeds 120 in.

- ◆ Use short-term modular ratio equal to 8 and long-term modular ratio equal to 24.
- ◆ Provide longitudinal slab reinforcement in accordance with *AASHTO LRFD Bridge Design Specifications*, Article 6.10.1.7.
- ◆ Assume the composite slab is effective in negative bending regions for Deflection check, Fatigue and Fracture Limit State, and Service Limit State.
- ◆ Do not use longitudinal slab reinforcement as part of the negative bending section for Strength Limit State.
- ◆ Unbraced flange length must satisfy either, Equation 6.10.1.6-2 or Equation 6.10.1.6-3.
- ◆ At flange splices, extend thicker flanges beyond the theoretical flange splice location by a length equal to the flange width but not more than 2 ft.

For stud connector designs, minimum longitudinal stud connector spacing is limited to $4d$, where d is the stud connector diameter.

Bolted field splices must meet the following requirements:

- ◆ Use of A 325 bolts is preferred over A 490 bolts.
- ◆ Assume Class A surface conditions. Class B surface conditions may be used only when slip controls the number of required bolts. Always note the surface condition assumed for design in the plans.
- ◆ Add at least 0.125 in., and preferably 0.25 in., to minimum edge distances shown in *AASHTO LRFD Bridge Design Specifications*, Table 6.13.2.6.6-1.
- ◆ Do not extend and develop fill plates equal to or thicker than 0.25 in. Instead, reduce bolt shear strength with *AASHTO LRFD Bridge Design Specifications*, Equation 6.13.6.1.5-1.

Section 13

Curved Plate Girders

Materials

Use A 709 Grade 50W steel for unpainted bridges. Use A 709 Grade 50 steel for painted bridges. You can use A 709 Grade HPS 70W steel for both unpainted and painted bridges if it is economical or otherwise beneficial to do so.

Use 0.875-in. or 1-in. diameter bolts for bolted connections.

Geometric Constraints

Minimum flange width is $0.30D$, where D =web depth, but not less than 15 in.

Minimum flange thickness is 1 in.

Minimum web thickness is 0.50 in.

Minimum stiffener thickness used to connect cross frames or diaphragms to girder is 0.50 in.

Structural Analysis

Beam designs must meet the following requirements:

- ◆ Distribute the weight of one railing to no more than three girders, applied to the composite cross section.
- ◆ Assume no slab haunch when determining composite section properties.
- ◆ A grid analysis or other refined analysis is required for curved girders. Curved girders satisfying *AASHTO LRFD Bridge Design Specifications*, Article 4.6.1.2.4b are excluded from this requirement. Use a single-lane-loaded multiple presence factor of 1.0.
- ◆ Use only one lane of live load in the structure model when checking the Fatigue and Fracture Limit State.

Design Criteria

Beam designs must meet the following requirements:

- ◆ Regarding *AASHTO LRFD Bridge Design Specifications*, Article 6.7.2, do not specify girders to be out-of-plumb in the steel-dead-load-only or theoretical-no-load condition. Diaphragms and cross frames have traditionally been installed with girders plumb and no significant problems have been reported to date. If the designer believes that analysis indicates that girders will

be significantly beyond plumb after slab concrete is placed, contact the Director of the Bridge Division for guidance.

- ◆ Diaphragm and cross-frame designs must meet the following requirements:
 - The maximum spacing is 20 ft. with curved girders if all limit states requirements are met.
 - Provide diaphragms/cross frames at all end bearings.
 - Place interior diaphragms/cross frames radial to girders. Do not use staggered placement of diaphragms/cross frames.
- ◆ Check the limiting slenderness ratio of cross-frame members using main compression member criteria provided in *AASHTO LRFD Bridge Design Specifications*, Article 6.9.3.
- ◆ Diaphragm and cross-frame members are primary members. Verify their adequacy for the Strength Limit State and other applicable limit states.

Girder designs must meet the following requirements:

- ◆ Use composite design and place shear connectors the full girder length.
- ◆ Do not use longitudinal stiffeners unless web depth exceeds 120 in.
- ◆ Use short-term modular ratio equal to 8 and long-term modular ratio equal to 24.
- ◆ Provide longitudinal slab reinforcement in accordance with *AASHTO LRFD Bridge Design Specifications*, Article 6.10.1.7.
- ◆ Assume the composite slab is effective in negative bending regions for Deflection check, Fatigue and Fracture Limit State, and Service Limit State.
- ◆ Do not use longitudinal slab reinforcement as part of the negative bending section for Strength Limit State.
- ◆ Unbraced flange length must satisfy either *AASHTO LRFD Bridge Design Specifications*, Equation 6.10.1.6-2 or Equation 6.10.1.6-3.
- ◆ At flange splices, extend thicker flanges beyond the theoretical flange splice location by a length equal to the flange width but not more than 2 ft.

For stud connector designs, minimum longitudinal stud connector spacing is limited to $4d$, where d is the stud connector diameter.

Bolted field splices must meet the following requirements:

- ◆ Use of A 325 bolts is preferred over A 490 bolts.
- ◆ Assume Class A surface conditions. Class B surface conditions may be used only when slip controls the number of required bolts. Always note the surface condition assumed for design in the plans.
- ◆ Add at least 0.125 in., and preferably 0.25 in., to minimum edge distances shown in *AASHTO LRFD Bridge Design Specifications*, Table 6.13.2.6.6-1.

- ◆ Do not extend and develop fill plates equal to or thicker than 0.25 in. Instead, reduce bolt shear strength with *AASHTO LRFD Bridge Design Specifications*, Equation 6.13.6.1.5-1.

Chapter 4

Substructure Design

Contents:

Section 1 — Overview

Section 2 — Foundations

Section 3 — Abutment

Section 4 — Rectangular Reinforced Concrete Bent Caps

Section 5 — Inverted Tee Reinforced Concrete Bent Caps

Section 6 — Columns for Multi-Column Bents

Section 1

Overview

Introduction

This chapter provides guidance on Load and Resistance Factor Design (LRFD) of specific bridge substructure components.

Section 2

Foundations

Guidance

Design foundations in accordance with requirements outlined in the TxDOT *Geotechnical Manual*, available at <ftp://ftp.dot.state.tx.us/pub/txdot-info/gsd/manuals/geo.pdf>

Section 3

Abutment

Materials

Use Class C concrete ($f'_c = 3.6$ ksi), and design for Grade 40 reinforcing steel, but allow use of Grade 40 or Grade 60 in the plans. Higher strengths may be required in special cases.

Geometric Constraints

For abutment supporting Type IV beams, Type VI beams, or U beams, use a cap width of at least 3 ft. 3 in. For all other I-beam types, use a cap width of 2 ft. 9 in. For other structure types refer to the bridge standard drawings for recommended cap widths.

Design Criteria

For calculating horizontal forces, use 40 lbs. per cu. ft. equivalent fluid pressure with 2 ft. of surcharge if no approach slab is used. Retaining type abutments in questionable soils may justify a more rigorous analysis.

Use the following design practice for standard type “stub” abutments with backwalls:

- ◆ Position the backwall, wing wall lengths, wing wall support, and various other standardized items as shown in the *Bridge Detailing Manual*, available at <http://crossroads.org/gsd/books/det/index.htm>, or applicable bridge standard drawings, available at <http://www.dot.state.tx.us/insdtdot/orgchart/cmd/cserve/standard/bridge-e.htm>.
- ◆ Cap, backwall, and wing wall reinforcing should conform to the *Bridge Detailing Manual*. Structural analysis is not required for abutments within the geometric constraints noted in the *Bridge Detailing Manual*.
- ◆ If no approach slab is used, calculate the horizontal forces using 40 pcf equivalent fluid pressure with a surcharge of $\Delta_p = k\gamma_s h_{eq}$, where $k = 0.25$, $\gamma_s = 120$ pcf.

For abutments with $d < 5$ ft. take $h_{eq} = 4.0$ ft. For all other abutments see AASHTO LRFD Table 3.11.6.4-1. Retaining type abutments in questionable soils may justify a more rigorous analysis.

- ◆ Provisions of *AASHTO LRFD Bridge Design Specifications*, Article 5.7.3.4 need not be satisfied. Limit spacing of primary flexural reinforcing bars to no more than 18 in.

For pile foundations, use battered pairs of piling for all abutments that are not otherwise restrained from horizontal movement. Examples of sufficient restraint are slab spans and pan form spans that are doweled into the abutment, and abutments within a mechanically stabilized fill. Never use battered piling adjacent to MSE walls because of the difficulty of installing the backfill.

The maximum spacing of drilled shafts or pile groups should not exceed 16 ft. with beams 40 in. and less deep nor 12.5 ft. with deeper beams.

Drilled shaft loads may be calculated as the total vertical load on the cap divided equally among the cap shafts. Wing wall shaft or pile load is usually taken as 10 tons per shaft or pile.

Calculate pile loads as the total vertical load on the cap divided equally among the cap pilings. For abutments with battered piling, add to the vertical load the load caused by 40-lb. per cu. ft. fluid pressure from the bottom of the cap to 2 ft. above the roadway surface. The back pile should not be allowed to go into tension due to the lateral load, considering dead load and soil pressure only.

Section 4

Rectangular Reinforced Concrete Bent Caps

Materials

Use TxDOT Class C concrete ($f'_c = 3.6$ ksi) and Grade 60 reinforcing steel. Higher concrete strengths may be required in special cases.

Geometric Constraints

Cap depth should be in 3-in. increments and not less than cap width.

For caps supporting Type IV beams, Type VI beam, or U beams, do not use a cap smaller than 3 ft 3 in. by 3 ft. 3 in. For all other I-beam types, do not use a cap smaller than 2 ft. 9 in. by 2 ft. 9 in. For other structure types refer to the bridge standard drawings for recommended cap widths.

Structural Analysis

In lieu of a more detailed analysis, analyze trestle pile and multiple-column caps as simply supported beams on knife-edge supports at the center of piling or columns. If the column is wide, consider a model that takes the stiffness of the column into consideration.

Distribute the live load to the beams assuming the slab hinged at each beam except the outside beam.

Base live load reactions per lane on the combined effect of the truck loading added to the lane loading.

Design Criteria

For cap design, check Strength I limit state and Service I limit state. Check distribution of reinforcement as required in Article 5.7.3.4 of the *AASHTO LRFD Bridge Design Specifications* using Class 1 exposure for moderate exposure conditions and Class 2 exposure in areas where deicing chemicals are frequently used. Limit tensile stress in steel reinforcement, f_{ss} under Service I limit state to $0.6f_y$.

Check Service I with dead load only, and limit reinforcement stress to 22 ksi to further minimize cracking.

For multi-column bent caps, take design negative moments at the center line of the column. For wide columns, take design negative moments at the effective face of the column.

Do not use the simplified procedures for determining shear resistance allowed by AASHTO LRFD 5.8.3.4.1 nor 5.8.3.4.3. Use the General Procedure as provided by AASHTO LRFD 5.8.3.4.2.

Minimize the number of stirrup spacing changes.

Except for hammerhead bents, shear need not be considered in cantilever regions unless the distance from center of load to effective face of column exceeds $1.2d$. Provide stirrups at 6-in. spacing.

For typical multi-column bent caps supporting beams on elastomeric bearing pads, strut-and-tie modeling provisions of Articles 5.6.3 need not be considered. For bent caps supporting girders on pot bearings or girders with large reaction forces that are defined as deep components according to Article 5.2, use the strut-and-tie design. Diagonal cracks have been observed in caps with large beam reactions and small pads designed using the traditional sectional model. Strut-and-tie analysis indicates that the diagonal compressive struts were overloaded in these instances.

When calculating the cracking moment of a member in accordance with Article 5.7.3.3.2, take the modulus of rupture, f_r , as $0.24\sqrt{f'_c}$, for all normal weight concrete.

Detailing

Use #5 stirrups with a 4-in. minimum and a 12-in. maximum spacing. Do not use stirrups larger than #6. Use double stirrups if required spacing is less than 4 in, otherwise.

For flexural reinforcement, use #11 bars. Smaller bars can be used to satisfy development requirements. Do not mix bar sizes.

Use longitudinal skin reinforcement in accordance with Equation 5.7.3.4-2 of the *AASHTO LRFD Bridge Design Specifications* in caps deeper than 3 ft. Caps 3 ft. and less should have two #5 bars equally spaced in each side face.

Section 5

Inverted Tee Reinforced Concrete Bent Caps

Materials

Use TxDOT Class C concrete ($f'_c = 3.6$ ksi) and Grade 60 reinforcing steel. Higher concrete strengths may be required in special cases.

Geometric Constraints

Make inverted tee dimensions the same for all bents on the project.

Keep the top of stem at least 2.5 in. below the bottom of the slab; see standard drawing IBMS, available at <http://www.dot.state.tx.us/insdtdot/orgchart/cmd/cserve/standard/bridge-e.htm>.

Structural Analysis

Analyze multiple-column caps as simply supported beams on knife-edge supports at the center of piling or columns. If the column is wide, consider a model that takes the stiffness of the column into consideration.

Distribute the live load to the beams assuming the slab hinged at each beam except the outside beam.

Design Criteria

For cap design, check Strength I limit state and Service I limit state. Check distribution of reinforcement as required in Article 5.7.3.4 of the *AASHTO LRFD Bridge Design Specifications* using Class 1 exposure for moderate exposure conditions and Class 2 exposure for areas where deicing chemicals are frequently used. Limit tensile stress in steel reinforcement, f_{ss} under Service I limit state to $0.6f_y$.

Check Service I with dead load only, and limit reinforcement stress to 22 ksi to further minimize cracking.

For multi-column bent caps, take design negative moments at the center line of the column. For wide columns, take design negative moments at the effective face of the column.

Minimize the number of stirrup spacing changes.

When designing for beam ledge punching shear, replace AASHTO LRFD Article 5.13.2.5.4 with the following:

- ◆ Use d_f , not d_e , in all ledge punching shear calculations.
- ◆ The truncated pyramids assumed as failure surfaces for punching shear, as illustrated in Figure 1, shall not overlap. Therefore, $0.5b_w + a_v$ must be greater than $0.5L + d_f$ to prevent longitudinal overlap. S must be greater than $2d_f + W$ to prevent transverse overlap.
- ◆ Normal punching shear resistance, V_n , kip, shall be taken as:

- At interior pads: $V_n = 0.125\sqrt{f'_c}(W + 2L + 2d_f)d_f$

- At exterior pads $V_n = 0.125\sqrt{f'_c}(0.5W + L + d_f + C)d_f$, except shall not be taken greater than $V_n = 0.125\sqrt{f'_c}(W + 2L + 2d_f)d_f$.

- ◆ Replace AASHTO LRFD Equation 5.13.2.5.5-1 with the following:

$$V_n = \frac{A_{hr}\left(\frac{2}{3}f_y\right)}{s}(W + 3a_v)$$

- ◆ Replace the following sentence in AASHTO LRFD Article 5.13.2.5.5: "The edge distance between the exterior bearing pad and the end of the inverted T-beam shall not be less than d_e " with the following: "The edge distance between the exterior bearing pad and the end of the inverted T-beam shall not be less than 12 inches."
- ◆ Replace the following sentence in AASHTO LRFD Article 5.13.2.5.5: "The hanger reinforcement specified herein shall be provided in addition to the lesser shear reinforcement required on either side of the beam reaction being supported" with the following: "Do not superimpose loads on stirrups acting as hangers and loads on stirrups acting as shear reinforcement. Proportion the web reinforcement in the stem of an invert T-beam based on required hanger reinforcement or required shear reinforcement, whichever is greater."

When calculating the cracking moment of a member in accordance with Article 5.7.3.3.2, take the modulus of rupture, f_r , as $0.24\sqrt{f'_c}$, for all normal weight concrete.

Detailing

Provide extra vertical reinforcing across the end surfaces of the stem to resist cracking. Do not weld bars together for development of ledge reinforcing. Use anchorage hooks to develop ledge reinforcing.

Use longitudinal skin reinforcement in accordance with Equation 5.7.3.4-2 of the *AASHTO LRFD Bridge Design Specifications* in caps deeper than 3 ft. Caps 3 ft. and less should have two #5 bars equally spaced in each side face.

Section 6

Columns for Multi-Column Bents

Materials

Use TxDOT Class C concrete ($f'_c = 3.6$ ksi) and design for Grade 40 reinforcing steel, but allow use of Grade 40 or Grade 60 in the plans. Higher concrete or steel strengths may be required in special cases.

Structural Analysis

Design is not necessary for round columns with the column diameter, typical reinforcement and recommended height limits shown in [this figure](#), as long as the column spacing is between 10 feet and 18 feet. Use the following diameters with the corresponding superstructures without analysis for axial and bending:

- ◆ Slab spans -- 24 in.
- ◆ Pan form spans -- 24 in.
- ◆ Types A and B and C prestressed beam spans -- 30 in.
- ◆ Types IV and VI prestressed beam spans and U-beam spans -- 36 in.
- ◆ For other beam types, compare drilled shaft load to what would be expected using one of the preceding superstructures, and use a column diameter as appropriate.

Chapter 5

Other Designs

Contents:

Section 1 — Widening

Section 2 — Steel-Reinforced Elastomeric Bearings for Prestressed Concrete Beams

Section 3 — Strut-and-Tie Method

Section 4 — Corrosion Protection

Section 1

Widenings

Design Recommendations

Design guidelines for various elements of new bridges may also be applied to bridge widenings. Complete a load rating and condition survey before plans are started. Ratings should be based on the 17th edition of the *American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges*, except that allowable stresses should be based on the minimum material strengths used on the original construction.

Refer to the *Bridge Project Development Manual* for additional requirements for load ratings and condition surveys.

Design widened portions for HL93 loading using the *AASHTO LRFD Bridge Design Specifications*.

Show load rating and design loads on the bridge plan, for example, HS20(Existing) and HL93(Widening).

Section 2

Steel-Reinforced Elastomeric Bearings for Prestressed Concrete Beams

Materials

Use 50-durometer neoprene for steel-reinforced elastomeric bearings.

Use a shear modulus range of 95 to 175 psi for design, using the least favorable value for the design check.

Make steel shims 0.105 in. thick.

Do not use adhesives between bearings and other components.

Geometric Constraints

See standard drawings available at <http://www.dot.state.tx.us/insdot/orgchart/cmd/cserve/standard/bridge-e.htm> for standard pad details.

Tapered bearings may be used if the taper does not exceed 0.055 ft./ft. For beams on steeper grades, use a beveled steel sole plate field-welded (1/4-in. fillet) to a 1/2-in. steel plate embedded in and anchored to beams with headed stud anchors. Use a minimum of four 1/2-in.-by-3-in. stud anchors with studs located between strands and reinforcement. The minimum thickness of sole plate should be 1.5 in. of steel between weld and elastomer. The sole plate should extend at least 1 in. beyond the beam flange. Sole plates should not be vulcanized to the bearing to allow slip to occur at the beam/bearing interface.

Use 1/4-in. exterior pad layers. If using 1/4-in. interior pad layers, disregard the requirements in the *AASHTO LRFD Bridge Design Specifications*, Article 14.7.6.1, specifying exterior layers no thicker than 70% of internal layers.

Structural Analysis

Assume a temperature change of 70 degrees Fahrenheit after erection when calculating thermal movement in one direction (not total). Take $T_{\min} = 10$ degrees F and $T_{\max} = 80$ degrees F. For the panhandle region use $T_{\min} = 10$ degrees F and $T_{\max} = 115$ degrees F, for a total temperature change of 105 degrees F.

Do not include shrinkage, creep, and elastic shortening when determining maximum movement, which will be accommodated through infrequent slip.

Do not apply *IM* to live load when checking compressive stress (see *AASHTO LRFD Bridge Design Specifications*, Commentary C14.7.5.3.2).

Use appropriate shear live load distribution, modified for skew.

Use the critical DL condition (the lightest predicted DL) when checking against slip.

Use Load Combination Service I for all gravity loads.

Design Criteria

Follow Design Method A in *AASHTO LRFD Bridge Design Specifications*, Article 14.7.6, with the following exceptions:

- ◆ DL compressive stress limit is the lesser of 1.20 ksi and 1.2 *GS*.
- ◆ Total compressive stress limit is the lesser of 1.50 ksi and 1.5 *GS*. This limit can be exceeded up to 15% at the engineer's discretion.
- ◆ For rotation check, disregard *AASHTO LRFD Bridge Design Specifications*, Article 14.7.6.3.5.

Rotation is acceptable if the total compressive deflection equals or exceeds $\frac{\theta(0.8L)}{2}$, where *L*

is the pad length defined in *AASHTO LRFD Bridge Design Specifications*, and θ is the total rotation. Estimate compressive deflection using *AASHTO LRFD Bridge Design Specifications*, Figure C14.7.5.3.3-1.

- ◆ Calculate total rotation for dead and live load plus 0.005 radians for construction uncertainties as required by *AASHTO LRFD Bridge Design Specifications*, Article 14.4.2.1. Take maximum

live load rotation as $\frac{4\Delta}{SpanLength}$, where Δ is midspan LL deflection.

- ◆ Check bearing pad slip as follows:

$$\Delta_{s(allow)} \leq \frac{(0.2 - Gr) \times DL \times h_{rt}}{(G \times A)}$$

where:

Gr = beam grade in ft./ft.

DL = lightest unfactored predicted dead load (kips)

h_{rt} = total elastomer thickness (M)

G = shear modulus of elastomer at 0 degrees F, typical 0.175 ksi

A = plan arc of elastomer (sq. in.)

$\Delta_{s(allow)}$ = maximum total allowable shear deformation (in.)

- ◆ You may use h_n , instead of total pad height when checking stability as required in *AASHTO LRFD Bridge Design Specifications*, Article 14.7.6.3.6.

Detailing

Use standard drawing IBEB for guidance on detailing custom bearing pad designs.

Section 3

Strut-and-Tie Method

Geometric Constraints

The angle between compression struts and tension ties must be greater than 26 degrees.

Structural Analysis

Strut and Tie Modeling shall not be used for standard girders and bent caps, but instead, for footings, dapped beam ends, post-tensioning anchorage zones, deviation diaphragms, bents that use high performance bearings, and other special designs.

Place nodes at applied loads and reactions. More nodes can be added as long as the tension ties are located where reinforcement is normally placed. The nodes should be located at the center of the tension ties and compression struts. If there is sufficient concrete in the incoming member the strut can be considered within both members, such as in the case with a column and a footing, and the nodes can be placed where the two members meet.

A 3 dimensional truss can be broken into multiple 2 dimensional trusses to be analyzed. When analyzing the 2 dimensional trusses, use the same reactions as the 3 dimensional truss, but recalculate the applied loads so equilibrium is satisfied.

Design Criteria

When designing broad members that are uniformly loaded on one end and discretely loaded on the other end, use *AASHTO LRFD Bridge Design Specifications*, Article 5.10.9.4.

Development length for the tension tie reinforcement must be in accordance with *AASHTO LRFD Bridge Design Specifications*, Articles 5.6.3.4.2 and 5.11.2.

Nodal geometry is needed to analyze the capacity of a strut and the capacity of the node itself; however, if the node is significantly large, such as with a column and a footing, no nodal analysis is needed. If nodal analysis is needed, use *AASHTO LRFD Bridge Design Specifications*, Article 5.6.3.5.

Replace AASHTO LRFD 5.6.3.3.3 with the following:

- ◆ For struts with reinforcement satisfying Section 5.6.3.6:

$$v = \frac{0.85 \tan \theta}{\sqrt{f'_c}} \frac{l_n}{w_s \sin \theta} \quad \text{but not greater than the minimum of}$$

$$0.85 \frac{l_n}{w_s \sin \theta} \quad \text{and } 0.85 \quad (\text{A-7})$$

- ◆ For struts without reinforcement satisfying Section 5.6.3.6:

$$v = \frac{0.85 \tan \theta}{3\sqrt{f'_c}} \frac{l_n}{w_s \sin \theta} \quad \text{but not greater than the minimum of}$$

$$0.85 \frac{l_n}{w_s \sin \theta} \quad \text{and } 0.85 \quad (\text{A-8})$$

The nominal capacity of a strut shall be taken as: $P_n = v f'_c A_c$

(A-9)

where:

P_n = nominal capacity of a strut (kip)

v = efficiency factor

f'_c = specified compressive strength (ksi)

A_c = cross-sectional area of the strut at the face of the node (in²)

θ = angle between the compressive strut and the adjoining tie (deg)

l_n = length of the node adjoining the strut. For CCC and CCT nodes $l_n = l_b$ and for CTT

nodes $l_n = l_a$ from Figure 5.6.3.3.2-1.

w_s = width of the strut at the face of the node (Figure 5.6.3.3.2-1)

Replace *AASHTO LRFD Bridge Design Specifications* Figure 5.6.3.3.2-1b "(b) Strut anchored by bearing and reinforcement" with the following figure:

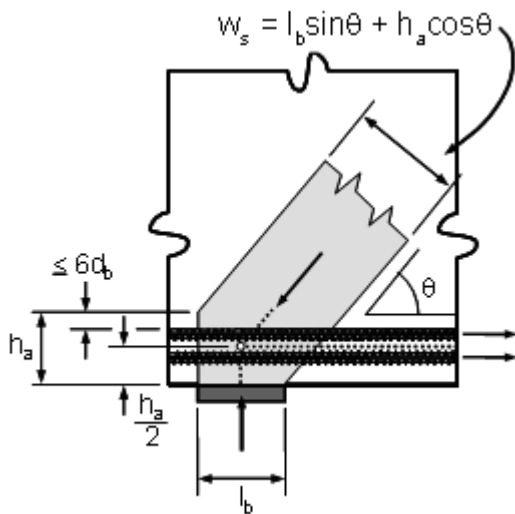


Figure 5-1.

Replace *AASHTO LRFD Bridge Design Specifications*, Figure 5.6.3.3.2-1 "(a) x-x" (cross section) with the following figure:

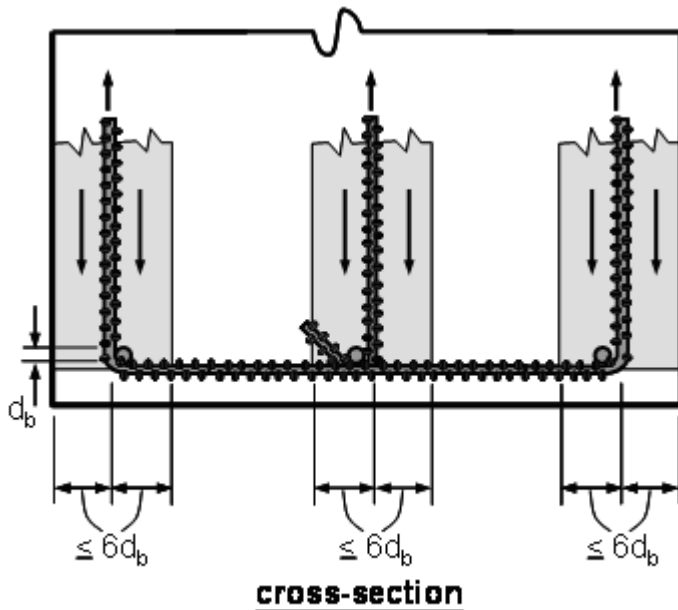


Figure 5-2.

Replace AASHTO LRFD 5.6.3.6 with the following:

- ◆ Structures and components or regions thereof, except for slabs and footings, which have been designed in accordance with the provisions of Article 5.6.3, and using the efficiency factor associated with reinforced struts (Equation A-7) shall contain crack control reinforcement in

an orthogonal grid. Horizontal reinforcement alone shall be used. The spacing of the bars in the strut reinforcement shall not exceed 12.0 in.

- ◆ The amount of reinforcement within a strut shall be calculated as:

$$\rho_{\perp} = \sqrt{\left(\frac{A_{sH}}{b \cdot s_H}\right)^2 + \left(\frac{A_{sV}}{b \cdot s_V}\right)^2} \quad (\text{A14})$$

Where:

ρ_{\perp} = equivalent reinforcement perpendicular to the strut axis

A_{sH} = total of horizontal reinforcement in a strut within spacing s_H (in²)

b = width of the member (in)

s_H = spacing of horizontal reinforcement (in)

A_{sV} = total area of vertical reinforcement in a strut within a spacing, s_V (in²)

s_V = spacing of vertical reinforcement (in)

- ◆ The minimum amount of reinforcement in a strut shall be taken as:

$$\rho_{\perp, \min} = \frac{P_u}{2f_y b l m} \geq 0.003 \quad (\text{A15})$$

Where:

P_u = factored load in a strut (kip)

f_y = yield strength of the reinforcement within a strut (ksi)

b = width of the member transverse to the plane of the strut-and-tie model

l = length of the strut

m = slope of the angle of compression dispersion (Figure A-2)

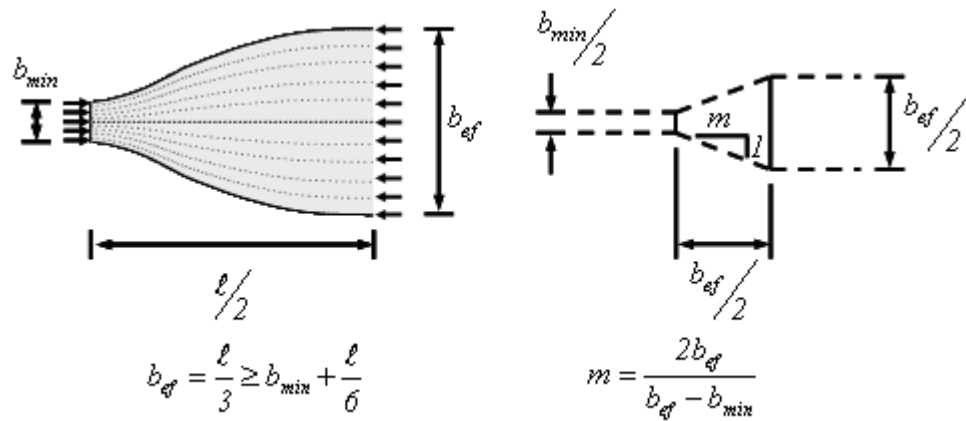


Figure 5-3.

Guidelines

The tension tie reinforcement must be close enough to the drilled shaft to be considered in the truss analysis. Therefore, the tension tie reinforcement must be within a 45 degree distribution angle (i.e. no more than d_c away from the member on either side).

Use strut bearing lengths proportional to the amount of load carried by the strut at a node.

Conservatively assume the width of a strut in a CCC node, h_s , as the height of the compression block.

Section 4

Corrosion Protection

Guidance

In areas of the state where deicing chemicals are frequently used during winter storms, it is recommended that additional corrosion protection measures be incorporated into the bridge design and details. Consult the Bridge Information web page for statewide and district specific recommendations.