Bridge Design Manual - LRFD

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Manual: *Bridge Design Manual - LRFD*

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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

Updates consist of the following: Chapter 1 includes the deletion of the Revision History Table. Chapter 2 is updated to include the Extreme Event III loading combination which relates to structural member or component failure. An additional section is added to Chapter 3, System Redundancy Evaluation for Steel Twin Tub Girders, requiring all newly steel twin tub girder bridges to be designed for system redundancy.

Supersedes

This revision supersedes version 2018-1.

Contact

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

Archives

Past Manual notices are available in a PDF archive.

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Chapter 1 — About this Manual

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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) mandated the use of LRFD for all bridges for which the Texas Department of Transportation (TxDOT) initiated 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*, 2017, 8th Edition, which designers should adhere to unless directed otherwise by this document.

All Articles, Equations, and Tables referenced in this manual are from the current AASHTO LRFD Bridge Design Specifications, unless noted otherwise.

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, steel-reinforced elastomeric bearings for pretensioned concrete beams, strut-and-tie method, and culverts. This chapter also addresses corrosion protection measures.

◆   Chapter 6, Archiving Design Notes. Policy for archiving bridge design notes in TxDOT's bridge inspection database management system.

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

Importance Factor

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 Article 1.3.5.

Redundancy

Add the following to Article 1.3.4 as follows:

$\eta_R \geq 1.05$ for nonredundant members and members requiring an analysis to establish system redundancy.

Extreme Event Limit States

Revise the following definition in Article 1.2 as follows:

Extreme Event Limit States—Limit states relating to events such as earthquakes, ice load, structural member or component failure, and vehicle and vessel collision, with return periods in excess of the design life of the bridge.

Revise Article 1.3.2.5 as follows:

Extreme Event Limit States – The extreme event limit state shall be taken to ensure the structural survival of a bridge during a major earthquake or flood, or when collided by a vessel, vehicle, or ice flow, possibly under scoured conditions, or after failure of a structural member or component.

Extreme Event I and II

Provisions under Extreme Event I need not be considered except for regions near Big Bend as noted in the subsequent section on Earthquake Effects.

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

Extreme Event III

Supplement Article 3.4.1 with the following:

◆ Extreme Event III - Load combination relating to a structural member or component failure.

Supplement Table 3.4.1-1 with the following:

Load Combination Limit State	DC DD DW EH EV ES PS CR SH	LL IM CE BR PL LS	WA	WS	WL	FR	TU	TG	SE	Use One of These at a Time				
										EQ	BL	IC	CT	CV
Extreme Event III	γ_p	1.10	1.00	--	--	1.00	--	--	--	--	1.00	1.00	1.00	1.00

Supplement Table 3.4.1-2 with the following:

Type of Load, Foundation Type, and Method Used to Calculate Downdrag	Load Factor	
	Maximum	Minimum
DC: Components and Attachments for the evaluation of system redundancy as specified in Chapter 3, Section 17, for Extreme Event III only	1.10	0.90

All load effects during an assumed fracture event due to both permanent and assumed transient loads shall be amplified by a factor of 1.20 to simulate the dynamic effects of a fracture on the twin tub girder span(s).

Multi-column Bridges

For typical multi-column bridges, determine design loads for foundations at Service I Load Combination. Determine foundation loads for single column bents and other non-typical substructures using Service I and Service IV Load Combinations. For Service IV Load Combination, 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 Article 3.6.1.1.2. Distribute the live load equally to all supporting foundations, assuming all lanes are loaded. Do not apply the Dynamic Load Allowance (IM) to the live load in determining foundation loads when the foundation elements are entirely below ground level.

Live Load Deflection

Check live load deflection using Articles 2.5.2.6.2 and 3.6.1.3.2. Ensure that the calculated deflection does not exceed 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 Span/1000.

Section 2 — Loads

Live Loads

Use HL93 design live load as described in Article 3.6.1.2 unless design for a special vehicle is specified or warranted.

Design widenings for existing structures using HL93. Rate existing structures using Load Factor Rating per the current *AASHTO Manual for Bridge Evaluation* 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 Article C3.6.1.1.2, 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.

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

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The provisions of Article 3.6.1.1.2 shall not be applied to the
extreme limit state when evaluating system redundancy as specified
in Chapter 3, Section 17 System Redundancy Evaluation for Steel
Twin Tub Girders.
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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.

Vehicular Collision Force

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—Do not investigate bents and piers for collision if the annual frequency for a bridge bent or pier to be hit by a vehicle, AF_{HPB}, is less than 0.001 using Equation C3.6.5.1-1. Design for vehicular collision force for the final condition after all construction is completed, not during construction phases with temporary traffic conditions. A bridge deck adjacent to the column is considered an adjacent roadway.

When investigating for collision, there are two design choices: redirect the collision load or provide structural resistance. When the design choice is to redirect the collision load, the protection must meet at least one of the following requirements:

◆ Protect with a structurally independent, ground-mounted 54-in. tall single slope concrete barrier (or other 54-in. tall, Test Level 5 approved barrier equivalent) if within 10 ft. from component.

◆ Protect with a 42-in. tall single slope concrete barrier (or 42-in tall, Test Level 5 approved barrier equivalent) if more than 10 ft. from component.

When the design choice is to provide structural resistance, design for the 600-kip equivalent static load as described in Article 3.6.5. See Chapter 4 - Substructure Design, Sections 6 - Columns for Multi-Column Bents and Section 7 - Columns for Single Column Bents or Piers for design information.

For structures with a clear distance of 25 ft. or less from the center line of a railway track, meet the requirements of American Railway Engineering and Maintenance-of-Way Association (AREMA), or the governing railroad company.

Earthquake Effects

Except as noted below, bridges and structures in Texas do not require analysis for seismic loading due to the low seismic hazard as shown in Article 3.10.2.

The TxDOT Bridge Standards and conventional bridge configurations have been evaluated for seismic effects and do not require further analysis.

For conventional structures with superstructure unit lengths or interior bent "H" heights outside of the limits stated in the TxDOT Bridge Standards and which are located in Brewster, Presidio, Jeff Davis, Culberson, Hudspeth and El Paso counties, check Minimum Support Length Requirements outlined in Article 4.7.4.4.

Non-conventional or exotic bridges do not require seismic evaluation, except those located in Brewster, Presidio, Jeff Davis, Culberson, Hudspeth and El Paso counties. In these locations, evaluate the structure for earthquake effects as required by Article 3.10. Contact the TxDOT Bridge Division for guidance.

Vessel Collision

TxDOT requires that all bridges crossing waterways with documented commercial vessel traffic comply with 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.

Pedestrian

Do not apply a pedestrian load to sidewalks when evaluating system redundancy at the Extreme Event III limit state.

Chapter 3 — Superstructure Design

Contents:

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 I-Girders, U-Beams, Spread Box Beams, Spread Slab Beams, Steel Plate Girders, and Steel Tub Girders

Materials

Use Class S concrete ($f_c^{'}$ = 4.0 ksi). Refer to district-specific corrosion protection requirements for regions where bridge decks are exposed to de-icing agents and/or saltwater spray with regularity. If thus required, use Class S (HPC) concrete.

Use Grade 60 reinforcing steel or deformed welded wire reinforcement (WWR) meeting the requirements of ASTM A1064. Refer to district-specific corrosion protection requirements for regions where bridge decks are exposed to de-icing agents and/or saltwater spray with regularity. If thus required, use one of the following types of corrosion resistant reinforcement (refer also to Item 440):

◆ Epoxy-Coated Reinforcing Steel meeting the requirements of ASTM A775 or A934

◆ Epoxy-Coated WWR meeting the requirements of ASTM A884 Class A or B

◆ Hot-Dip Galvanized Reinforcing Steel

◆ Glass Fiber Reinforced Polymer (GFRP) Bars; The design for GFRP reinforcement in bridge decks must adhere to the AASHTO LRFD Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings.

◆ Dual Coated Reinforcing Steel meeting the requirements of ASTM A1055

◆ Low Carbon/Chromium Reinforcing Steel meeting the requirements of ASTM A1035 Gr 100 Ty CS

◆ Stainless Reinforcing Steel meeting the requirements of ASTM A955 Ty 316LN, XM-28, 2205, or 2304; Use only for extreme chloride exposure in coastal areas.

Geometric Constraints

TxDOT standard deck slab is 8.5 in. deep. Use of thinner concrete decks is not permitted.

Cover to reinforcing bars is 2.5 in. clear to the top mat and 1.25 in. clear to the bottom mat. Cover to bar ends is 2 in.

Maximum overhang is 3.33 ft. beyond the design section for negative moment specified in Article 4.6.2.1.6, but not more than 1.3 times the girder depth.

Minimum overhang is 0.5 ft. from top beam or flange edge except for spread slab and spread box beams, which have a 0 ft. minimum overhang.

Design Criteria

Where applicable, use the Empirical Design of Article 9.7.2 with the following exceptions:

◆ Top mat reinforcement is No. 4 bars at 9 in. maximum spacing (0.27 sq. in./ft.) in both transverse and longitudinal direction. Place longitudinal bars closest to the top slab surface. In the overhangs, place No. 5 bars extending 2 ft. minimum past fascia girder web centerline between each transverse No. 4 bar.

◆ Bottom mat reinforcement is No. 4 bars at 9 in. maximum spacing (0.27 sq. in./ft.) in both transverse and longitudinal directions. Place transverse bars closest to the bottom slab surface.

◆ Slab regions adjacent to expansion joints are reinforced as shown on the standard drawings depicting thickened slab end details. No additional reinforcement in end regions, including those skewed over 25°, is needed in these cases. These standard drawings are:
  - IGTS for Tx Girders
  - UBTS for U-beams
  - SGTS for Steel I-beams and Plate Girders
  - XBTS for Spread Box Beams

◆ Cross-frames or diaphragms are not needed at supports for any prestressed concrete beam or girder.

◆ Do not provide supplemental reinforcement over the webs of U-beams or steel tub girders.

◆ The deck does not need to be fully cast-in-place and can utilize stay-in-place concrete formwork such as prestressed deck panels shown on standard drawing PCP.

◆ The overhang need not extend past the exterior girder more than 6 in. beyond the flange edge (0 in. for spread slab or spread box beams). An overhang is not required for girders and beams for the temporary condition of having a stage or phase construction joint located on top of their flange.

Use the Traditional Design of Article 9.7.3 where the provisions listed above for empirical deck use are not met. The minimum amount of longitudinal reinforcement in the top mat is No. 4 bars at 9 in. maximum spacing for these deck designs.

Overhang strength for extreme events, per Article 9.5.5, is satisfied through TxDOT's rail crash testing.

Detailing

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

Section 3 — Concrete Deck Slabs on Adjacent-Framed Beams (Slab Beams and Box Beams)

Materials

Use Class S concrete ($f_c^{'}$ = 4.0 ksi). Refer to district-specific corrosion protection requirements for regions where bridge decks are exposed to de-icing agents and/or saltwater spray with regularity. If thus required, use Class S (HPC) concrete.

Use Grade 60 reinforcing steel or deformed welded wire reinforcement (WWR) meeting the requirements of ASTM A1064. Refer to district-specific corrosion protection requirements for regions where bridge decks are exposed to de-icing agents and/or saltwater spray with regularity. If thus required, use one of the following types of corrosion resistant reinforcement (refer also to Item 440):

- Epoxy-Coated Reinforcing Steel meeting the requirements of ASTM A775 or A934

- Epoxy-Coated WWR meeting the requirements of ASTM A884 Class A or B

- Hot-Dip Galvanized Reinforcing Steel

- Glass Fiber Reinforced Polymer (GFRP) Bars; The design for GFRP reinforcement in bridge decks must adhere to the AASHTO LRFD Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings.

- Dual Coated Reinforcing Steel meeting the requirements of ASTM A1055

- Low Carbon/Chromium Reinforcing Steel meeting the requirements of ASTM A1035 Gr 100 Ty CS

- Stainless Reinforcing Steel meeting the requirements of ASTM A955 Ty 316LN, XM-28, 2205, or 2304; Use only for extreme chloride exposure in coastal areas.

Geometric Constraints

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

Use 2.5-in. top clear cover.

Design Criteria

For transverse reinforcement, use #5 bars spaced at 6 in. maximum.

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

Detailing

Place transverse reinforcement parallel to the skew for skews up to 30°.

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

Section 4 — Pretensioned Concrete I Girders

Materials

Use Class H concrete with a minimum $f_{ci}' = 4.0$ ksi and $f_c' = 5.0$ ksi and a maximum $f_{ci}' = 6.0$ ksi and $f_c' = 8.5$ ksi.

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

Geometric Constraints

The minimum number of I-girders in any roadway width is four if the span is over a lower roadway and the vertical clearance is less than 20 feet. Otherwise, a minimum of three I-girders per span may be used.

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

Structural Analysis

Girder designs must meet the following requirements:

◆   Distribute the weight of one railing to no more than three girders, applied to the composite cross section.

◆   Use section properties given on the Prestressed Concrete I-Girders standard drawings.

◆   Composite section properties may be calculated assuming the girder and slab to have the same modulus of elasticity (for girders with $f_c' < 8.5$ ksi). Do not include haunch concrete placed on top of the girder when determining section properties. Section properties based on final girder and slab modulus of elasticity may also be used; however, this design assumption must be noted on the plans.

◆   Live load distribution factors must conform to Article 4.6.2.2.2 for flexural moment and Article 4.6.2.2.3 for shear, except as noted below:

   ●   For exterior girder design with a slab cantilever length equal to or less than one-half of the adjacent interior girder spacing, treat the exterior girder as if it were an interior girder to determine the live load distribution factor for the interior girder. The slab cantilever length is defined as the distance from the centerline of the exterior girder to the edge of the slab.

   ●   For exterior girder design with a slab cantilever length exceeding one-half of 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 beam must never be less than the live load used to design an interior beam of comparable length.

◆   Do not use the special analysis based on conventional approximation for loads on piles per Article C4.6.2.2.2d, unless the effectiveness of diaphragms on the lateral distribution of truck loads is investigated.

◆   Do not take the live load distribution factor for moment or shear as less than the number of lanes divided by the number of girders, including the multiple presence factor per Article 3.6.1.1.2.

◆   When prestressed concrete deck panels or stay-in-place metal forms are allowed, design the beam using the basic slab thickness.

Design Criteria

Standard girder designs must meet the following requirements:

◆   Add and drape strands in the order shown on the standard drawing IGND.

◆   Straight strand designs with and without debonding are permitted provided stress and other limits noted below are satisfied.

◆   Debonded strands must conform to Article 5.9.4.3.3 except as noted below:

●   The maximum debonding length is the lesser of: (a) one-half the span length minus the maximum development length; (b) 0.2 times the beam length; or (c) 15 ft.

●   Not more than 75% 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).

◆   Use hold-down points shown on the standard drawing IGD.

◆   Strand stress after seating of chucks is limited to $0.75f_{pu}$ for low-relaxation strands.

◆   Initial tension stress up to $0.24\sqrt{f'_{ci}}$ (ksi) is allowed for all standard TxDOT I-girder sections.

◆   Initial compression stress up to $0.65f_{ci}'$ (ksi) is allowed.

◆   Final stress at the bottom of girder 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 stress up to $0.19\sqrt{f_c'}$ (ksi) is allowed.

◆   The required final concrete strength (f_c') is typically based on compressive stresses, which must not exceed the following limits:

● $0.60f_c'$ for stresses due to total load plus effective prestress.

● $0.45f_c'$ for stresses due to effective prestress plus permanent (dead) loads.

- $0.40 f_c'$ for stresses due to Fatigue I live loads plus one-half of the sum of stresses due to prestress and permanent (dead) loads.

◆ Tension stress up to $0.24 \sqrt{f_c'}$ is allowed for checking concrete stresses during deck and diaphragm placement.

◆ Use an effective strand stress after release of $0.75 f_{pu} - \Delta f_{pES}$

◆ Keep the end position of depressed strands as low as possible so that the position of the strands does not control the release strength. Release strength can be controlled by end conditions when the depressed strands have been raised to their highest possible position.

◆ Use the General Procedure as provided by Article 5.7.3.4.2 to determine shear resistance. Do not use provisions of Appendix B5 of the *AASHTO LRFD Bridge Design Specifications*.

◆ Calculate required stirrup spacing for #4 Grade 60 bars according to the Article 5.7. Change stirrup spacing as shown on standard drawing IGD for I girders only if analysis indicates the inadequacy of the standard design.

◆ Replace Equation 5.7.4.5-1 with the following:

$$v_{ui} = \frac{V_{u1} Q_{slab}}{I_g b_{vi}}$$

where Q_{slab} is the first moment of the area of the slab with respect to the neutral axis of the composite section.

Take b_{vi}, width of the interface, equal to the beam top flange width. Do not reduce b_{vi} to account for prestressed concrete panel bedding strips.

◆ Determine interface shear transfer in accordance with Article 5.7.4. Take cohesion and friction factors as provided in Article 5.7.4.4 as follows:

$c = 0.28$ ksi

$\mu = 1.0$

$K_1 = 0.3$

$K_2 = 1.8$ ksi

◆ Replace Equation 5.4.2.3.2-2 with the following:

$k_s = 1.45 - 0.13 \, (V/S) > 0.0$

◆ Compute deflections due to slab weight and composite dead loads assuming the girder and slab to have the same modulus of elasticity. Assume $E_c = 5,000$ ksi for girders with $f_c' < 8.5$ ksi. Show predicted slab deflections 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-girders reinforced as shown on the IGD standard drawings are adequate for the requirements of Article 5.9.4.4.

◆   A calculated positive (upward) camber is required after application of all permanent (dead) loads.

Use the following equations to determine prestress losses:

◆   Total prestress losses: $\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR}$

◆   Elastic shortening:

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp}, \text{ where } f_{cgp} = 0.7 f_{pu} A_{ps} \left(\frac{1}{A_g} + \frac{e_p^2}{I_g} \right) - \frac{M_g e_p}{I_g}$$

◆   Shrinkage loss: $\Delta f_{pSR} = E_p \left(\dfrac{140 - H}{4.8 + f'_{ci}} \right) 4.4 * 10^{-5}$

◆   Creep loss: $\Delta f_{pCR} = 0.1 \left(\dfrac{195 - H}{4.8 + f'_{ci}} \right) \left(\dfrac{E_p}{E_{ci}} \right) (f_{cgp} + 0.6 \Delta f_{cd})$,

   where $\Delta f_{cd} = -\left(\dfrac{M_{sd} e_p}{I_g} \right)$

◆   Relaxation loss: $\Delta f_{pR} = \dfrac{2 f_{pt}}{K_L} \left(\dfrac{f_{pt}}{f_{py}} - 0.55 \right)$

Use of *AASHTO LRFD Bridge Design Specifications 2004, 3rd Ed.*, Article 5.9.5, Loss of Prestress, is also allowed (available from the Bridge Division). Other methods to determine prestress losses are not allowed.

Section 5 — Pretensioned Concrete U Beams

Materials

Use Class H concrete with a minimum $f_{ci}' = 4.0$ ksi and $f_c' = 5.0$ ksi and a maximum $f_{ci}' = 6.0$ ksi and $f_c' = 8.5$ ksi.

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

Geometric Constraints

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

Structural Analysis

Beam designs must meet the following requirements:

◆ 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.

◆ Calculate composite section properties 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; however, this design assumption must be noted on the plans.

◆ Live load distribution factors for interior beams must conform to 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 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 better live load distribution arising from the beneficial torsional stiffness of the box girder system.

● 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 of comparable length.

- 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 Article 3.6.1.1.1.

- Do not take the live load distribution factor for moment or shear as less than the number of lanes divided by the number of girders, including the multiple presence factor per Article 3.6.1.1.2.

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 Article 5.9.4.3.3 except as noted below:
  - Debond no more than 75% of the total number of strands.
  - Debond no more than 75% of the number of strands in that row.
  - The maximum debonding length is the lesser of: (a) one-half the span length minus the maximum development length; (b) 0.2 times the beam length; or (c) 15 ft.
  - Not more than 75% 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 Section 4, Pretensioned I-Girders for other design criteria.

Detailing

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

Section 6 — Pretensioned Concrete Slab Beams

Materials

Use Class H concrete with a minimum $f_{ci}' = 4.0$ ksi and $f_c' = 5.0$ ksi and a maximum $f_{ci}' = 6.0$ ksi and $f_c' = 8.5$ ksi.

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° without modification to standard drawings.

The minimum gap between adjacent slab beams is 0.5 in. and the maximum gap is 3.31 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 Slab Beams 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; however, this design assumption must be noted on the plans.

◆ Live load distribution factors for all beams, both moment and shear, must conform to 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.

◆ Do not take the live load distribution factor for moment or shear as less than the number of lanes divided by the number of girders, including the multiple presence factor per Article 3.6.1.1.2.

Design Criteria

Standard beam designs must meet the following requirements:

◆ Add strands in the order shown on the PSBND standard drawing.

◆ Debond strands in 3-ft. increments at beam ends if necessary to control stresses at release.

◆ Debonded strands must conform to Article 5.9.4.3.3 except as noted below:

  ● Debond no more than 75% of the total number of strands.

  ● Debond no more than 75% of the number of strands in that row.

  ● The maximum debonding length is the lesser of: (a) one-half the span length minus the maximum development length; (b) 0.2 times the beam length; or (c) 15 ft.

  ● Not more than 75% 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 Article 5.7. Change stirrup spacing as shown on relevant standard drawings only if analysis indicates inadequacy of the standard design.

◆ TxDOT standard slab beams satisfy Article 5.7.4 and Article 5.9.4.4.

◆ 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 ksi 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 4, Pretensioned Concrete I-Girders](#) for other design criteria.

Section 7 — Pretensioned Concrete Spread Slab Beams

Materials

Use Class H concrete with a minimum $f_{ci}' = 4.0$ ksi and $f_c' = 5.0$ ksi and a maximum $f_{ci}' = 6.0$ ksi and $f_c' = 8.5$ ksi.

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

Geometric Constraints

The maximum skew angle for spread slab beam bridges is 30° without modification to standard drawings.

Target slab overhangs at 0 ft. past beam edge. Minimal overhangs to accommodate roadway curvature are acceptable.

Structural Analysis

Beams 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 Decked Slab Beams standard drawings.

◆ Composite section properties may be calculated assuming the beam and composite slab 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; however, this design assumption must be noted on the plans.

◆ Live load distribution factors for shear and moment are available from the Bridge Division.

◆ 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 take the live load distribution factor for moment or shear as less than the number of lanes divided by the number of girders, including the multiple presence factor per Article 3.6.1.1.2.

◆ When prestressed concrete deck panels or stay-in-place metal forms are allowed, design the beam using the basic slab thickness.

Design Criteria

Standard beam designs must meet the following requirements:

◆ Add and debond strands in the order shown on the PSBND standard drawings.

◆ Debond strands in 3-ft. increments at beam ends if necessary to control stresses at release.

◆ Debonded strands must conform to Article 5.9.4.3.3 except as noted below:

- Debond no more than 75% of the total number of strands.

- Debond no more than 75% of the number of strands in that row.

- The maximum debonding length is the lesser of: (a) one-half the span length minus the maximum development length; (b) 0.2 times the beam length; or (c) 15 ft.

- Not more than 75% 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 Article 5.7. Change stirrup spacing as shown on relevant standard drawings, only if analysis indicates the inadequacy of the standard design.

◆ TxDOT standard slab beams satisfy Article 5.7.4 and Article 5.9.4.4.

◆ 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 ksi 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 4, Pretensioned Concrete I-Girders for other design criteria.

Detailing

Modify standard drawings for slab beams by extending composite steel (Bars H) above the top of the beams to reflect use of 8.5 in. thick decks. The standard drawings base their composite steel extension on use of 5 in. thick decks required for adjacently framed slab beams.

Section 8 — Pretensioned Concrete Decked Slab Beams

Materials

Use Class H concrete with a minimum $f_{ci}' = 4.0$ ksi and $f_c' = 5.0$ ksi and a maximum $f_{ci}' = 6.0$ ksi and $f_c' = 8.5$ ksi.

Use non-shrink cementitious grout for shear keys.

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

Geometric Constraints

The maximum skew angle for decked slab beam bridges is 30°.

Use a 2-in. minimum thickness asphaltic concrete pavement (ACP) overlay for most roadways. A two-course surface treatment, or no wearing surface, may be used for low-volume roadways at the District's discretion.

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 Decked Slab Beams standard drawings.

◆ Live load distribution factors for all beams (both moment and shear) must conform to Table 4.6.2.2b-1, using cross section (j) if beams are connected only enough to prevent relative vertical displacement at their interfaces. Use $K = 2.0$ when determining the live load distribution factor.

◆ Use $S/10$ as maximum limit on live load distribution.

◆ Do not apply the skew correction factor for moment as suggested in Article 4.6.2.2.2e.

◆ Do not take the live load distribution factor for moment or shear as less than the number of lanes divided by the number of girders, including the multiple presence factor per Article 3.6.1.1.2.

Design Criteria

Standard beam designs must meet the following requirements:

◆ Add and debond strands in the order shown on the DSBND standard drawings.

◆ Debond strands in 3-ft. increments at beam ends if necessary to control stresses at release.

　● Debond no more than 75% of the strands per row or per section.

　● The maximum debonding length is the lesser of: (a) one-half the span length minus the maximum development length; (b) 0.2 times the beam length; or (c) 15 ft.

◆ Calculate required stirrup spacing for #4 Grade 60 bars according to the Article 5.7. Change stirrup spacing as shown on the DSBD standard drawing, only if analysis indicates the inadequacy of the standard design.

◆ Standard decked slab beams satisfy Article 5.7.4 and Article 5.9.4.4.

◆ Connect adjacent beams with lateral connectors, shown on standard drawing DSBD, spaced at 5 ft. maximum, with the first lateral connectors set 1 ft. from bent centerlines. See span standard drawings for completion of lateral connection details.

◆ See Section 4, Pretensioned Concrete I-Girders, for other design criteria.

Section 9 — Pretensioned Concrete Box Beams

Materials

Use Class H concrete with a minimum of $f_{ci}^{'} = 4.0$ ksi and $f_c^{'} = 5.0$ ksi and a maximum $f_{ci}^{'} = 6.0$ ksi and $f_c^{'} = 8.5$ ksi.

Use Class S concrete ($f_c^{'} = 4.0$ ksi) for shear keys.

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° without modification to standard drawings.

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

A 5-in. minimum thickness composite concrete slab overlay is required. A 2-in. minimum thickness asphaltic concrete pavement (ACP) overlay can be used in lieu of the concrete deck for low-volume roadways at the discretion of the District.

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 Box Beams 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; however, this design assumption must be noted on the plans.

◆ Live load distribution factors must conform to 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.

◆ Do not take the live load distribution factor for moment or shear as less than the number of lanes divided by the number of girders, including the multiple presence factor per Article 3.6.1.1.2.

Design Criteria

Standard beam designs must meet the following requirements:

◆   Add and debond strands in the order shown on the BBND standard drawings.

◆   Debond strands in 3-ft. increments at beam ends if necessary to control stresses at release.

◆   Debonded strands must conform to Article 5.9.4.3.3 except as noted below:

   ●   Debond no more than 75% of the total number of strands.

   ●   Debond no more than 75% of the number of strands in that row.

   ●   The maximum debonding length is the lesser of: (a) one-half the span length minus the maximum development length; (b) 0.2 times the beam length; or (c) 15 ft.

   ●   Not more than 75% 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 Article 5.7. Change stirrup spacing as shown on relevant standard drawings only if analysis indicates the inadequacy of the standard design.

◆   TxDOT standard box beams satisfy Article 5.7.4 and Article 5.9.4.4.

◆   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 $E_c = 5,000$ ksi 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, available from the Bridge Division on request.

◆   See Section 4, Pretensioned Concrete I-Girders, for other design criteria.

Section 10 — Pretensioned Concrete Spread Box Beams (X-Beams)

Materials

Use Class H concrete with a minimum of $f_{ci}' = 4.0$ ksi and $f_c' = 5.0$ ksi and a maximum $f_{ci}' = 6.0$ ksi and $f_c' = 8.5$ ksi.

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

Geometric Constraints

The maximum skew angle for X-beam bridges is 30° without modification to standard drawings.

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 X-Beams standard drawings.

◆   Composite section properties may be calculated assuming the beam and composite slab 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; however, this design assumption must be noted on the plans.

◆   Live load distribution factors for interior beams must conform to 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 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 better live load distribution arising from the beneficial torsional stiffness of the box girder system.

   ●   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.

◆ Do not take the live load distribution factor for moment or shear as less than the number of lanes divided by the number of girders, including the multiple presence factor per Article 3.6.1.1.2.

◆ When prestressed concrete deck panels or stay-in-place metal forms are allowed, design the beam using the basic slab thickness.

Design Criteria

Standard beam designs must meet the following requirements:

◆ Add and debond strands in the order shown on the XBND standard drawings.

◆ Debond strands in 3-ft. increments at beam ends if necessary to control stresses at release.

◆ Debonded strands must conform to Article 5.9.4.3.3 except as noted below:

  ● Debond no more than 75% of the total number of strands.

  ● Debond no more than 75% of the number of strands in that row.

  ● The maximum debonding length is the lesser of:

    ● one-half the span length minus the maximum development length;

    ● 0.2 times the beam length;

    ● 15 feet.

  ● Not more than 75% 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 Article 5.7. Change stirrup spacing as shown on relevant standard drawings, only if analysis indicates the inadequacy of the standard design.

◆ TxDOT standard X-beams satisfy Article 5.7.4 and Article 5.9.4.4.

◆ 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 ksi 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 4, Pretensioned Concrete I-Girders, for other design criteria.

Section 11 — Cast-in-Place Concrete Slab and Girder Spans (Pan Forms)

Materials

Use Class S concrete ($f_c^{'}$ = 4.0 ksi). Refer to district-specific corrosion protection requirements for regions where bridge decks are exposed to de-icing agents and/or saltwater spray with regularity. If thus required, use Class S (HPC) concrete.

Use Grade 60 reinforcing steel. Refer to district-specific corrosion protection requirements for regions where bridge decks are exposed to de-icing agents and/or saltwater spray with regularity. If thus required, use one of the following types of corrosion resistant reinforcement (refer also to Item 440):

◆ Epoxy-Coated Reinforcing Steel meeting the requirements of ASTM A775 or A934

◆ Hot-Dip Galvanized Reinforcing Steel

◆ Glass Fiber Reinforced Polymer (GFRP) Bars; The design for GFRP reinforcement in bridge decks must adhere to the AASHTO LRFD Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings.

◆ Dual Coated Reinforcing Steel meeting the requirements of ASTM A1055

◆ Low Carbon/Chromium Reinforcing Steel meeting the requirements of ASTM A1035 Gr 100 Ty CS

◆ Stainless Reinforcing Steel meeting the requirements of ASTM A955 Ty 316LN, XM-28, 2205, or 2304; Use only for extreme chloride exposure in coastal areas.

Geometric Constraints

The only skew angles and span lengths available for pan form spans are provided on the Concrete Slab and Girder (Pan Form) standard drawings. Forming systems currently in use do not provide for alternative skews or span length.

Limit slab overhangs to a maximum of 13.75 in. 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 12 — Cast-in-Place Concrete Slab Spans

Materials

Use Class S concrete ($f_c' = 4.0$ ksi). Refer to district-specific corrosion protection requirements for regions where bridge decks are exposed to de-icing agents and/or saltwater spray with regularity. If thus required, use Class S (HPC) concrete.

Use Grade 60 reinforcing steel or deformed welded wire reinforcement (WWR) meeting the requirements of ASTM A1064. Refer to district-specific corrosion protection requirements for regions where bridge decks are exposed to de-icing agents and/or saltwater spray with regularity. If thus required, use one of the following types of corrosion resistant reinforcement (refer also to Item 440):

◆ Epoxy-Coated Reinforcing Steel meeting the requirements of ASTM A775 or A934

◆ Epoxy-Coated WWR meeting the requirements of ASTM A884 Class A or B

◆ Hot-Dip Galvanized Reinforcing Steel

◆ Glass Fiber Reinforced Polymer (GFRP) Bars; The design for GFRP reinforcement in bridge decks must adhere to the AASHTO LRFD Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings.

◆ Dual Coated Reinforcing Steel meeting the requirements of ASTM A1055

◆ Low Carbon/Chromium Reinforcing Steel meeting the requirements of ASTM A1035 Gr 100 Ty CS

◆ Stainless Reinforcing Steel meeting the requirements of ASTM A955 Ty 316LN, XM-28, 2205, or 2304; Use only for extreme chloride exposure in coastal areas.

Geometric Constraints

The maximum skew angle for slab span bridges is 30°. With skewed spans, use shear keys that are 2 in. deep by 6 ft. wide and parallel to traffic. Form shear keys into the top of substructure caps in the middle of the caps. See the Cast-In-Place Concrete Slab Spans standard drawings for shear key details.

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

Minimum slab depths from Table 2.5.2.6.3-1 are guidelines but are not required.

Use a top clear cover of 2.5 in. 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 Article 3.6.1.3.3 for spans more than 15 ft.

Distribute live load in accordance with 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 Articles 5.12.2.1 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 Article 4.6.2.3.

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

Provide bottom transverse distribution reinforcement. Use Equation 5.12.2.1-1 to determine the required amount.

Provide #4 reinforcing bars at 12-in. maximum spacing for shrinkage and temperature reinforcement required to satisfy Article 5.10.6.

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.

Section 13 — Straight Plate Girders

Materials

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

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

For bridges in the Amarillo District only, specify tension components to meet Zone 2 tension component impact test requirements.

Geometric Constraints

Minimum flange width is $0.20D$, 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

Girder 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 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.

◆   Do not take the live load distribution factor for moment or shear as less than the number of lanes divided by the number of girders, including the multiple presence factor per Article 3.6.1.1.2.

◆   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 girder designs must meet the following requirements:

◆   Specify fit condition in the plans when necessary as recommended in Article 6.7.2, and specify steel dead load fit (SDLF) where possible.

◆   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 interior diaphragms/cross-frames parallel to bents or abutments for skews up to 20°. Set interior diaphragms/cross-frames perpendicular to girders for skews beyond 20°.

   ●   Check the limiting slenderness ratio of cross-frame members using criteria provided in Articles 6.8.4 and 6.9.3.

   ●   Lean-on bracing design, as described in Cross-Frame and Diaphragm Behavior for Steel Bridges with Skewed Supports, Helwig and Wang, Research Report 1772-1, 2003, is permissible.

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 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. When calculating stresses in structural steel for composite sections in negative bending for the Service II Limit state, only include the concrete deck in the section properties if tensile stress in the deck is less than $2f_r$ per Article 6.10.4.2.1.

◆   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.

◆ Include an assumed stay-in-place formwork weight of 15 psf in design.

◆ Specify continuous placement of bridge deck where possible, and staged placement only if required. If staged placement is specified, base girder design on the worst case effect of staged and continuous placement. Base dead load deflection and camber on an analysis for continuous placement, even if staged placement is specified in the plans.

◆ For stud connector designs, minimum longitudinal stud connector spacing is limited to 4*d*, where *d* is the stud connector diameter. For structures with web depths greater than 24", disregard 6.10.10.1.2 and limit stud connector spacing to a maximum of 24 in.

Provide bolted field splices as the primary method of field splicing in the plans. Include the weight of the splice plates in the steel weight for payment. Bolted field splices must meet the following requirements:

◆ Use ASTM F3125 Grade A325 bolts. Use galvanized Grade A325 bolts for painted structures. Use Grade A490 bolts only if the connection cannot be designed with A325 bolts. Do not specify galvanized Grade A490 bolts for any structure.

◆ 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 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 Equation 6.13.6.1.4-1.

Section 14 — Curved Plate Girders

Materials

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

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

For bridges in the Amarillo District only, specify tension components to meet Zone 2 tension component impact test requirements.

Geometric Constraints

Minimum flange width is 0.25D, 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

Girder 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 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

Girder designs must meet the following requirements:

- ◆ Specify fit condition in the plans when necessary as recommended in Article 6.7.2, and specify steel dead load fit (SDLF) where possible.

- ◆ 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 primary member criteria provided in Articles 6.8.4 and 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 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. When calculating stresses in structural steel for composite sections in negative bending for the Service II Limit state, only include the concrete deck in the section properties if tensile stress in the deck is less than $2f_r$ per Article 6.10.4.2.1.

- ◆ 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.

- ◆ Include an assumed stay-in-place formwork weight of 15 psf in design.

- ◆ Specify continuous placement of bridge deck where possible, and staged placement only if required. If staged placement is specified, base girder design on the worst case effect of staged and continuous placement. Base dead load deflection and camber on an analysis for continuous placement, even if staged placement is specified in the plans.

- ◆ For stud connector designs, minimum longitudinal stud connector spacing is limited to $4d$, where d is the stud connector diameter. For structures with web depths greater than 24", disregard 6.10.10.1.2 and limit stud connector spacing to a maximum of 24 in.

Provide bolted field splices as the primary method of field splicing in the plans. Include the weight of the splice plates in the steel weight for payment. Bolted field splices must meet the following requirements:

- Use ASTM F3125 Grade A325 bolts. Use galvanized Grade A325 bolts for painted structures. Use Grade A490 bolts only if the connection cannot be designed with A325 bolts. Do not specify galvanized Grade A490 bolts for any structure.

- 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 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 Equation 6.13.6.1.4-1.

Section 15 — Segmental Spans

Materials

Use TxDOT Class H concrete with a minimum f_c' = 5.0 ksi, and Grade 60 reinforcing steel.

Use 0.6-in., low-relaxation prestressing strand with a specified tensile strength, f_{pu} of 270 ksi.

Use duct as specified in Item 426, "Post-Tensioning" of the Texas Standard Specifications.

Geometric Constraints

Use minimum cross-section dimensions as specified in Article 5.12.5.3.

For maintenance, the clear height of interior box section should not be less than 6 feet.

Structural Analysis

Analyze segmental spans in accordance with Article 4.6.2. Analysis must consider time-dependent construction methods; short- and long-term post-tensioning, creep, and shrinkage effects; and secondary force effects due to post-tensioning.

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

Round prestress losses to 1 ksi.

Design Criteria

Check limit states using the Strength I, Strength III, Strength IV, Service I, and Service III load combinations.

In addition to the limit states and loads specified in Chapter 2 and above, apply the construction loads and combinations in Article 5.12.5. Check the cantilever for overturning during erection as specified in Article 5.12.5.4.4.

Satisfy all stress limits for prestressing tendons and concrete as required in Articles 5.9.2.2 and 5.9.2.3. Use limits for severe corrosive conditions in areas of the state where de-icing agents are frequently used.

For all considerations other than preliminary design, determine prestress losses as specified in Article 5.9.3, including consideration of the time-dependent construction method and schedule shown in the contract documents.

Check principal stress in webs as required in Article 5.9.2.3.3.

Check shear and torsion as required in Article 5.12.5.3.8.

Determine tie back reinforcement behind anchorages as required in Article 5.9.5.6.7b. Minimize the amount of tie back reinforcing steel penetrating through segment bulkheads by adjusting the location of anchorages or using hooks for development.

Overhang strength for extreme events, described in Article 9.5.5, is satisfied through TxDOT's rail crash testing.

Calculate prestress losses for creep, shrinkage, elastic shortening and relaxation as prescribed in Section 4, Pretensioned Concrete I Girders, or by analysis software that has concrete time-dependent capabilities to capture the effect of creep and shrinkage.

Detailing

Provide 2 in. clear cover to reinforcing steel for entire cross section. Add 1 in. grinding allowance to top slab. Also, increase top slab clear cover to 2.5 in. in areas of state where de-icing agents are frequently used.

Include shear keys in webs of precast and cast-in-place segmental spans. Space shear keys 1 ft. 6 in. along centerline of web.

Provide a minimum 5 ft. tangent length of tendon from the anchorage head before introducing any curvature. Determine minimum radius of curvature for individual duct sizes based on published values from suppliers.

Include extra longitudinal reinforcement above a horizontal construction joint to mitigate cracking from restraint shrinkage.

Include extra longitudinal reinforcement above a doorway opening or other significant cross-section change, to mitigate cracking from differential shrinkage.

Include provisional post-tensioning ducts as required in Article 5.12.5.3.9. Provisional anchorage hardware is not required, but allowance for future placement of hardware must be considered.

Provide a ¾" deep continuous drip-bead adjacent to deck coping as shown IGMS.

Provide access openings at maximum 600 feet spacing and the distance from any location of box girder to the nearest opening should not be more than 300 feet. Provide at least two openings per box girder line. The size of access opening should be 32 in. x 42 in. or 36 in. diameter at minimum.

Section 16 — Spliced Precast Girders

Materials

Use Class H (HPC) concrete for girder elements:

◆   Precast Elements:

◆   Minimum $f_{ci}^{'}$ = 4.0 ksi, Maximum $f_{ci}^{'}$ = 6.0 ksi

◆   Minimum $f_c^{'}$ = 5.0 ksi, Maximum $f_c^{'}$ = 10.0 ksi

◆   Cast in Place Elements:

◆   Maximum $f_c^{'}$ = 6.0 ksi

Use Class S concrete for cast in place deck (or Class S (HPC) if de-icing chemicals are routinely used at the site):

◆   Maximum $f_c^{'}$ = 4.0 ksi

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

◆   Use 0.6 in low-relaxation strands for pretensioning strands.

◆   Use 0.6 in low-relaxation strands for post-tensioning tendons.

Provide post tension system in accordance with Item 426, "Post Tensioning" of the TxDOT Standard Specifications, with the following exceptions:

◆   Non-Severe Corrosive Environments:

   ●   Galvanized or plastic duct can be used

   ●   Meet requirements for Protection Level 1B

   ●   Do not use tape-sealed connections

◆   Severe Corrosive Environments:

   ●   Use plastic duct only

   ●   Meet requirements for Protection Level 2

All stressed tendons in the finished structure must be grouted. All tendons that are stressed at the precast yard must be grouted prior to transport.

Geometric Constraints

The minimum numbers of girders in any roadway width is as follows:

◆   I-Section: 3 girders. If the span is over a lower roadway and the vertical clearance is less than 20 ft., a minimum of 4 girders are required.

◆   U-Section: 2 girders.

Structural Analysis

Girder designs should meet the following requirements:

◆   Use a web or flange thickness that is a whole number and results in the duct not exceeding 0.54 times the least gross concrete thickness at the duct.

◆   Distribute the weight of one railing to no more than three girders.

◆   Haunch concrete placed on top of the girder may be considered when determining composite section properties.

◆   Composite section properties can be calculated assuming either constant modulus of elasticity for the girders and slab, or transforming the sections based upon their respective modulus. Determine respective modulus in accordance with AASHTO LRFD Bridge Design Specifications, Article 5.4.2.4.

◆   Live load distribution can be determined from one of the following methods:

•   Must conform to AASHTO LRFD Bridge Design Specifications, Article 4.6.2.2.2 for flexure moment and Article 4.6.2.2.3 for shear when used in conjunction with a line girder analysis.

•   As determined by use of the lever rule when the span/girder arrangement is out of the applicable range of Articles 4.6.2.2.2 and 4.6.2.2.3 when used in conjunction with a line girder analysis.

•   As distributed by the model when used in conjunction with a grillage, finite element, or other refined model. The model must capture the effects of the complete unit and transfer loads in an acceptable fashion.

◆   When prestressed concrete deck panels or stay-in-place metal forms are allowed, design the girder using the basic slab thickness.

Analysis must consider the effects of the following:

◆   Staged construction

◆   Addition and removal of temporary supports

◆   Locked in forces

◆   Staged post tensioning

◆   Secondary forces due to post tensioning

◆   Torsion due to horizontally curved alignments

- ◆ Superstructure / Substructure interaction

- ◆ Temperature variation

Design Criteria

Provide a minimum of two tendons per web.

Use diaphragms at all bearing locations.

Provide a full depth diaphragm at all splice and anchorage locations.

Intermediate diaphragm use is not mandatory.

The precast sections must meet the following requirements for transportation:

- ◆ Prestressed Sections:
  - • Factor the self-weight load by 1.33
  - • Strand stress after seating of chucks is not greater than $0.75 f_{pu}$ for low-relaxation strands
  - • Use the concrete release strength $(f_{ci}^{'})$ for the following stress limitations:
    Tensile stress $< 0.24\lambda \sqrt{f'_{ci}}$ (ksi)
    Compressive stress $< 0.65 f_{ci}^{'}$ (ksi)
  - • Do not drape pretensioning strands. Debond the strands as needed.

- ◆ Non-Prestressed Sections:
  - • Factor the self-weight load by 1.33
  - • Design the section as a reinforced concrete member, subject to the provisions in *AASHTO LRFD Bridge Design Specifications,* Article 5.6.3. Use the concrete release strength $(f_{ci}^{'})$ in place of the concrete final strength $(f_{c}^{'})$
  - • Limit the stress in the reinforcing steel to 36 ksi

The precast sections must meet the following requirements during construction stages:

- ◆ Factor the self-weight load by 1.0

- ◆ Include loads to represent weight of form work for splices and strong backs (if applicable)

- ◆ Use the final concrete strength $(f_{c}^{'})$ for the following stress limitations:
  Tensile stress $< 0.24\lambda \sqrt{f'_{c}}$ (ksi)
  Compressive stress $< 0.6 f_{c}^{'}$ (ksi)

The girder must meet the following requirements in the final (service) condition.

- ◆ Use associated final concrete strengths $(f_{c}^{'})$ for the precast sections and cast in place splices

◆ Use effective prestress force after all short and long term losses. Losses can be determined by hand as outlined in Section 4, Pretensioned Concrete I Beams and I Girders, or by analysis software that has concrete time dependent capabilities to capture the effect of creep and shrinkage.

◆ Compressive stress limitations:

    ● Service I Loading $< 0.6 f_c'$

    ● Effective Prestressing and Permanent (Dead) Loading $< 0.45 f_c'$

◆ Tensile stress limitations:

    ● Service III Loading

        Non-Severe Corrosive Environment $< 0.19\lambda \sqrt{f'}_c$ (ksi) ≤ 0.6ksi

        Severe Corrosive Environment $< 0.09\lambda \sqrt{f'}_c$ (ksi) ≤ 0.3ksi

    ● Effective Prestressing and Permanent (Dead) Loading – No tension allowed

◆ Principal Tensile stress at Neutral Axis of Web

    Service III Loading $< 0.110\lambda \sqrt{f'}_c$ (ksi)

All post tensioning must be done prior to placement of the deck.

The composite deck is not a prestressed element and is not held to the stress limitations listed above.

The deck must meet the following requirements:

◆ Design Load includes effects due to the following:

    ● Pouring sequence

    ● Superimposed loads applied to composite section of Service III. Exclude the effects of creep and shrinkage of deck concrete.

◆ Longitudinal steel must meet the following requirements:

    ● Tensile stress in deck concrete is less than $(0.9)(0.24)\lambda \sqrt{f'}_c$ (ksi), use No. 4 bars at 9-in spacing

    ● Tensile stress in deck concrete is greater than $(0.9)(0.24)\lambda \sqrt{f'}_c$ (ksi), deck reinforcement must equal or exceed 1% of the gross deck cross-sectional area (do not use bars larger than No. 6):

Design shear based upon Strength I Loading for the final condition and in accordance with Article 5.7.3.3 of the AASHTO LRFD Bridge Design Specifications. Use the General Procedure as provided by Article 5.7.3.4.2. Do not use provisions of Appendix B of the AASHTO LRFD Bridge Design Specifications. The effective web width must be reduced by 25% of the outer diameter of the splice coupler.

Design ultimate moment based upon Strength 1 Loading for the final condition.

Refer to Section 4, Pretensioned Concrete I Beams and I Girders for interface shear design of the deck to girder flange interface.

Show predicted slab deflections should be shown on the plans. Compute deflections using the same composite sections (constant modulus for girder and deck, or transformed sections) used in the analysis. Denote on plans the assumed modulus (if constant is used) or the assumed values of f_c' of the individual elements.

Included in plans the assumed construction sequence that includes the following:

◆   Order of construction

◆   Shore tower locations

◆   Shore tower loads

◆   Lifting / support points of precast members

◆   Girder elevation points

◆   Post tensioning sequence

Require contractor to provide a temporary bracing plan of the girders.

Require contractor to provide shoring and erection plan.

Detailing

Provide 2-in. clear cover to reinforcing steel for entire cross section. Also, increase top slab clear cover to 2.5 in. in areas of state where de-icing agents are frequently used.

Provide a minimum tangent length, dependent on duct size and type, of tendon from the anchorage head before introducing any curvature. Determine minimum radius of curvature for individual duct sizes based on published values from suppliers.

Reference Item 426 "Post Tensioning" in the General Notes for all post tensioning, grouting materials, and construction. Note exceptions if Protection Level 1B is used in the design (galvanized duct allowed)

Provide anchorage zone details per Article 5.9.5.6

Section 17 — `System Redundancy Evaluation for Steel Twin Tub Girders`

Structural Analysis

All two tub girder bridges must satisfy the requirements in this manual and must be evaluated for system redundancy of spans at the Extreme Event Limit State III as described in Chapter 2. Two types of analysis can be used to evaluate the Extreme Event III:

◆ **Approximate structural analysis**, as described in *Modeling the Response of Fracture Critical Steel Box-Girder Bridges*, Barnard et al., Research Report 5498-1, 2010 and the **Simplified Method** as described in the TxDOT Bridge Design Guide, for two tub girder bridges is permitted when:

- Spans do not exceed 250 ft

- Supports are skewed no more than 20 degrees

- Horizontal curvature greater than 700 ft

- Engineer ascertains that the use of an approximate analysis method is adequate.

For the approximate analysis to be permitted for spans satisfying the conditions specified above, the entire self-weight of the span under consideration and the entire live load shall be assumed carried by the intact girder after the assumed fracture event. It shall also be assumed that prior to fracture, the fractured girder was carrying 50% of the total dead load and the entire live load on the bridge, and thus it shall be assumed that the bridge slab must transfer this load from the fractured girder to the intact girder.

◆ **Refined structural analysis**, as described in *Modeling the Response of Fracture Critical Steel Box-Girder Bridges*, Barnard et al., Research Report 5498-1, 2010, shall account for the capacity of the intact girder as well as portions of the fractured girder that can still provide structural resistance, such as interior support locations. The load distribution between the intact girder and the fractured girder shall be realistically modeled. A table of live load distribution coefficients for extreme force effects in each span is not required when evaluating system redundancy as specified in Chapter 3, Section 17 System Redundancy Evaluation for Steel Twin Tub Girders.

A structurally continuous railing, barrier, or median, acting compositely with the supporting components, may be considered to be structurally active at Extreme Limit State III when evaluating system redundancy as specified in Chapter 3, Section 17 System Redundancy Evaluation for Steel Twin Tub Girders.

Design Criteria

General

These provisions shall only apply for the evaluation of the system redundancy of spans with twin tub-girder cross-sections at the Extreme Event III Limit State. For the purposes of these provisions, the applicable Extreme Event III load combination specified in the modified Table 3.4.1-1 in Chapter 2, shall apply.

Twin Tub-girder spans satisfying the system redundancy requirements of Chapter 3 Section 17 System Redundancy Evaluation for Steel Twin Tub Girders shall be assumed to possess adequate system redundancy at Extreme Event III Limit State. Members or portions within such spans that would otherwise be classified as fracture critical when evaluated based on load path redundancy alone, shall instead be designated in the contract documents as SRMs (system redundant members) and need not be subject to the hands-on in-service inspection protocol for fracture critical members (FCMs) as described in 23 CFR 650. The SRMs shall be fabricated according to the American Welding Society (AWS) D1.5 Bridge Welding Code Fracture Control Plan (FCP).

For Extreme Event III Limit State for Twin Tub Plate Girder Bridges, investigation for system redundancy shall be limited to end spans of continuous units and all simple spans.

One girder within the span under consideration shall be assumed to be fractured while the other girder in the same span and both girders in all remaining spans shall be assumed to remain fully intact. The bottom flange in tension and the webs attached to that flange of the fractured girder shall be assumed to be fully fractured at the location of the maximum factored tensile stress in the bottom flange determined using the Strength I load combination. To result in the worst case loading scenario, the girder assumed to be fractured shall be chosen based on its position in the cross-section relative to the traffic lanes and its eccentricity to the deck and railing. If the span under consideration is horizontally curved, the girder with the largest radius should be assumed to be the fractured girder.

Live Load

The HL-93 live load, including both truck and lane load, shall be positioned on the bridge deck directly above the presumed fracture location to cause the most severe internal stresses to develop in the assumed intact girder. The number, width, and location of design lanes shall be taken as the number, width, and location of striped traffic lanes on the bridge.

Internal and External Diaphragms

Internal and external diaphragms shall be provided at all supports. These diaphragms and their connections to the boxes shall be designed to resist the torsional moment in the assumed intact girder, and to transmit vertical and lateral forces to the bearings during and after an assumed fracture event. These diaphragms shall also be designed to act compositely with the slab with the

shear connectors designed as specified in this Section under the below subsection Shear, Shear Connectors.

Additionally, at least two permanent external intermediate diaphragms, designed according to AASHTO and Extreme Event III, shall be provided on each side of the location of the maximum factored tensile stress in the bottom flange in the span under consideration determined using the Strength I load combination. These two permanent external diaphragms should be located no further than a distance of 0.1 to 0.2 of the span length from the location of maximum factored tensile stress in the bottom flange and shall each be placed in-line with an internal intermediate diaphragm or cross-frame. These diaphragms should be as deep as practicable, but as a minimum should be at least 0.75 times the tub-girder depth. The permanent external intermediate diaphragms need not be designed to act compositely with the slab and their flanges need not be connected to the tub-girder flanges.

Connections

Bolted slip-critical connections in twin tub-girder spans shall also be proportioned to provide shear, bearing, and tensile resistance in accordance with Articles 6.13.2.7, 6.13.2.9, and 6.13.2.10, as applicable, at the Extreme Event III limit state when evaluating the span for system redundancy as specified in Chapter 3, Section 17. Standard holes or short-slotted holes normal to the line of force shall be used in such connections.

Flexure

The intact tub girder and portions of the fractured girder that can still resist load shall be checked for adequate flexural resistance after the assumed fracture event under Extreme Event III load combination according to the provisions of Article 6.11.7 and 6.11.8, as applicable.

Shear

The intact tub girder and portions of the fractured girder that can still resist load shall be checked for adequate shear resistance after the assumed fracture event under Extreme Event III load combination according to the provisions of Article 6.11.9. St. Venant torsional shears shall be included in the calculation of V_u, where applicable.

◆ **Concrete Deck** - The concrete deck shall be checked for adequate shear resistance to resist the shear due to torsion after the assumed fracture event under the Extreme Event III load combination according to the provisions of Article 5.7.3.3. The use of empirical deck design as described in Article 9.7.2 is prohibited.

◆ **End Diaphragms** - End diaphragms and their connection to both tub girders shall be checked to ensure adequate resistance to the torque applied to the intact girder after the assumed fracture event under Extreme Event III load combination.

◆ **Shear Connectors** - Stud shear connectors connecting the deck to the assumed fractured girder shall have sufficient tension capacity to develop the plastic beam mechanism in the

bridge deck after the assumed fracture event. In lieu of an acceptable alternative approach, these shear connectors and the shear connectors on all support diaphragms shall be designed for combined shear and axial force according to the provisions of Article 6.16.4.3. As an alternative, the analysis method for shear connectors from *Modeling the Response of Fracture Critical Steel Box-Girder Bridges*, Barnard et al., Research Report 5498-1, 2010 is permissible. This alternative approach neglects shear on the studs in the fractured girder due to the assumption that the fractured girder is not carrying any load. All shear connectors shall be detailed to extend above the bottom mat of deck reinforcement.

◆ **Top Flange Lateral Bracing** - Top flange lateral bracing can be considered part of the resisting section for St. Venant torsional shears in addition to the concrete deck. The contributions of the deck and top lateral bracing are additive.

Detailing

Use the following detailing criteria when designing Twin Tub-Girder Bridges for system redundancy:

◆ All details on both tub girders, with the exception of drain holes in the bottom flange and details on the bracing members shall have a fatigue resistance based on Detail Category C′ or higher. Drain holes in the bottom tension flange shall be located at least 20 ft. from the location of the maximum tensile stress in the flange determined using the Strength I load combination.

◆ Positive restraint and adequate support lengths shall be provided to keep the superstructure on the substructure after the assumed fracture event. Bearings need not be evaluated for this limit state.

◆ Structurally continuous barrier railings at least 32 in. in height shall be provided and should be considered to be structurally active for the analysis at the Extreme Event III limit state as permitted in Chapter 3, Section 17 System Redundancy Evaluation for Steel Twin Tub Girders.

Submittal and Approval

To satisfy FHWA requirements, TxDOT Bridge Division must approve each steel twin tub girder bridge design for system redundancy. At the 60% PS&E level, in coordination with the District Bridge Engineer, send a pdf of the following documents to the Bridge Division Design Section Director for Bridge Division approval.

◆ Bridge Layout

◆ Steel twin tub girder plan sheets

◆ Steel twin tub girder design calculation package

An approval memo will be sent to the District and filed in the bridge inspection management system.

Upon TxDOT's acceptance and approval of 100% Plans, submit calculations and full completed refined analysis records/computer models as this information will be retained and included with the bridge inspection management system.

Chapter 4 — Substructure Design

Contents:

Section 1 — Overview

Introduction

This chapter documents policy 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 *Geotechnical Manual*.

Design foundations to be in compression under Service I Load Combination. Exceptions are permitted only where additional foundation elements and/or repositioning foundation elements cannot prevent tension in the foundation elements under Service I Load Combination. If foundations are in tension in the service or factored limits states, including structures with significant staged construction foundation variations, provide structural details that ensure adequate load transfer throughout the substructure.

Design foundations and substructures for changes in foundation conditions due to scour as noted in Article 3.7.5. Refer to *Hydraulic Manual* and *Geotechnical Manual* for additional guidance on design for scour.

The beneficial effects of system behavior incorporating bents of varying configurations and abutment resistance may be used to evaluate the capacity of foundations and substructures.

Monoshafts framing into to dissimilarly sized columns must conform to the non-contact splice conditions of Article 5.10.8.4.2. For monoshafts framing with single column bents, perform analysis to examine the consequences of deflection under lateral loads.

Section 3 — Abutments

Materials

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

Geometric Constraints

For abutments supporting Tx70 girders, use a cap width of at least 4.00 ft. and 42 in. diameter drilled shafts as a minimum.

For abutments supporting Type IV beams or U beams, use a cap width of at least 3.25 ft. and 36 in. diameter drilled shafts as a minimum. For all other structure types refer to the bridge standard drawings for recommended cap widths.

Design Criteria

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 Guide*, or applicable bridge standard drawings. In unique cases requiring additional bearing area, the primary backwall may be positioned at the back of the abutment cap.

◆   Minimum reinforcement for cap, backwall, and wing wall reinforcing needs is shown in the *Bridge Detailing Guide*. Structural analysis is generally not required for abutments within the geometric constraints noted in the *Bridge Detailing Guide*. Exceptions include cases where abutment has significant likelihood of acting as a bent due to scour and stream migration or known future expansion plans would result in lengthening the bridge.

◆   Provisions of Article 5.6.7 need not be satisfied for abutment caps not requiring analysis as noted above. Limit spacing of primary flexural reinforcing bars to no more than 18 in.

◆   Calculate the horizontal forces using 40 pcf equivalent fluid pressure at the bottom of the cap. If no approach slab is used, include 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 Table 3.11.6.4-1. Retaining type abutments in questionable soils may justify a more rigorous analysis.

For pile foundations, use battered pairs of piling for all abutments that are not otherwise restrained from horizontal movement or otherwise consistent with standard abutment designs/details shown on standard drawings. Examples of sufficient restraint are slab spans and pan form spans that are doweled into the abutment. If analysis determines adequate resistance to lateral loads, vertical pile

abutments in MSE wall backfill are permitted for deeper girders than the standard abutment designs/details shown on standard drawings. Avoid battered piling in areas immediately adjacent to MSE walls because of the difficulty of installing the backfill. If sufficient room is provided for MSE wall straps and compaction, battered piles may be used.

Maximum spacing of drilled shafts or pile groups:

- TxGirders less than or equal to 40 in. in depth - 13.50 ft.

- TxGirders greater than 40 in. in depth - 11.00 ft.

- Steel girders greater than 70 in. in depth - 11.00 ft unless lesser spacing is required by analysis

- All other beam types less than or equal to 40 in. in depth - 16.00 ft.

- All other beam types greater than 40 in. in depth - 12.50 ft.

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, unless calculated vertical loads are higher.

Calculate pile loads as the total vertical load on the cap divided equally among the cap pilings. For abutments with battered piling, add the horizontal force specified above to the vertical load. The back pile is not allowed to go into tension due to the lateral load, considering dead load and soil pressure only unless the addition of further foundation elements or other mitigation efforts fail to eliminate the presence of tension.

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.

Higher reinforcing steel grades may be used provided their use satisfies requirements in AASHTO.

Geometric Constraints

Cap depth cannot be less than cap width unless the cap is widened for the purposes of:

◆   meeting minimum support length per Article 4.7.4.4, or

◆   accommodating a cap to column connection when one or both elements are precast, or

◆   satisfying vertical clearance needs to a lower roadway.

For bents supporting Tx70 girders, use a cap width of at least 4.00 ft. and 42 in. columns and drilled shafts as a minimum.

For bents supporting U40 or U54 beams, use a cap width of at least 3.25 ft. and 36 in. columns and drilled shafts as a minimum.

For all other structure types refer to the bridge standard drawings for minimum cap widths.

Structural Analysis

In lieu of a more detailed analysis, it is permissible to 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 wider than 4 ft., 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

Check limit states using the Strength I and Service I load combinations. Check distribution of rein-forcement as required in Article 5.6.7 using Class 1 exposure for moderate exposure conditions and

Class 2 exposure in areas where de-icing agents are frequently used or where contact with salt spray is possible. Limit tensile stress in steel reinforcement, f_{ss} under Service I load combination to $0.6 f_y$.

Check Article 5.6.3.3 for minimum reinforcement.

For reinforced concrete straddle bents, check the calculated shear, using the Service I Load Combination, against the resistance from Equation C5.8.2.2-1.

For multi-column bent caps, take design negative moments at the center line of the column. For hammerhead bents and multi-column bent caps with columns 4 ft. wide or wider, take design negative moments at the effective face of the column.

Minimize the number of stirrup spacing changes.

Ignore requirements in Article 5.7.3.2 requiring that concentrated loads located within d_v from the face of support, the shear load and shear resistance shall be calculated at the face of the support. Loads close to the support are transferred directly to the support by compressive arching action without causing additional stresses in the stirrups.

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. maximum spacing.

For typical multi-column bent caps supporting multiple beams, strut-and-tie modeling provisions of Articles 5.8.2 need not be considered. For bent caps supporting girders on high load multi rotational (HLMR) bearings or girders with large reaction forces that are defined as deep components according to Article 5.2, use the strut-and-tie design.

Detailing

Use #5 stirrups except as noted, 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. If torsional resistance is explicitly addressed in the design, ensure the stirrup detailing is consistent with AASHTO requirements.

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 5.6.7-3 in caps deeper than 3 ft. Caps 3 ft. and less should have two #5 bars, as a minimum, 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.

Higher reinforcing steel grades may be used provided their use satisfies requirements in AASHTO.

Geometric Constraints

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

Keep the top of stem at least 2.5 in. below the bottom of the slab; see standard drawing IGMS.

Structural Analysis

In lieu of a more detailed analysis, it is permissible to analyze multiple-column caps as simply supported beams on knife-edge supports at the center of piling or columns. If the column is wider than 4 ft., 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

Check limit states using the Strength I and Service I load combinations. Check distribution of reinforcement as required in Article 5.6.7 using Class 1 exposure for moderate exposure conditions and Class 2 exposure for areas where de-icing agents are frequently used or where contact with salt water spray is possible.

Check Article 5.6.3.3 for minimum reinforcement.

For reinforced concrete straddle bents, check the calculated shear, using the Service I Load Combination, against the resistance from Equation C5.8.2.2-1.

For multi-column bent caps, take design negative moments at the center line of the column. For hammerhead bents and multi-column bent caps with columns 4 ft. wide or wider, take design negative moments at the effective face of the column.

Minimize the number of stirrup spacing changes.

The punching shear resistance and hanger reinforcement provided at fascia girders must equal or exceed the factored punching shear demand and hanger reinforcement requirements of the adjacent interior girder.

◆ Replace Equation 5.8.4.3.5-1 with the following:

$$V_n = \frac{A_{hr}\left(\frac{2}{3}f_y\right)}{s}(W + 3a_v),$$ with f_y not taken larger than 60 ksi.

◆ ** OR ** Replace Equation 5.8.4.3.5-1 with the following:

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

with f_y not taken larger than 60 ksi.

◆ The edge distance between the exterior bearing pad and the end of the inverted T-beam stem shall not be less than 12 inches.

Detailing

Provide extra vertical reinforcing across the end surfaces of the stem to resist cracking. Single #5 bars, anchored at each end with hooks and with 6-in. (+/-) spacing, are considered adequate for this purpose for conventional inverted tee cap ends. Do not weld bars together for development of ledge reinforcing. Use anchorage hooks to develop ledge reinforcing.

Use stirrups with a 12-in. maximum spacing. If torsional resistance is explicitly addressed in the design, ensure the stirrup detailing is consistent with AASHTO requirements.

Section 6 — Columns for Multi-Column Bents

Materials

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

Higher reinforcing steel grades may be used, provided their use satisfies requirements in AASHTO.

Geometric Constraints

The minimum size column and drilled shaft for grade separation structures is 36-in diameter unless a larger size is noted elsewhere. Column and drilled shaft sizes smaller than 36-in diameter are permissible for widenings only for similitude.

Structural Analysis

Analysis and design is not required for round columns supporting multi-column bents when the following conditions are met:

◆ Column spacing does not exceed 18 ft.

◆ Column height, measured from bottom of the cap to top of drilled shaft or footing, does not exceed 12 times the column diameter (measured in feet). Example: 36' height limit for a 3-ft diameter column. For drilled shaft foundations in stream crossings, the bottom of the column is to be taken at the bottom of the scour envelope.

◆ Columns are reinforced with the minimum amount of reinforcement, both longitudinally and laterally, as prescribed in AASHTO.

◆ Columns meet these size requirements based on superstructure type:

  ● Slab spans, 24 in. for stream crossings, 36 in. for grade crossings.

  ● Pan form spans, 24 in. for stream crossings, 36 in. for grade crossings.

  ● Slab beam and spread slab beam spans, 24 in. for stream crossings, 36 in. for grade crossings.

  ● Box beam and spread box beam spans, 36 in.

  ● Types Tx28 through Tx54 girder spans, 36 in.

  ● Types Tx62 and Tx70 girder spans, 42 in.

  ● U40 and U54 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.

If these conditions are not met, column design and analysis, including second order effects and stiffness reduction from cracked concrete, is required.

Design Criteria

For columns subjected to bending under unfactored dead load, satisfy the minimum reinforcement requirements of Article 5.6.7, using an exposure factor consistent with the site and other bridge elements.

Detailing

For non-contact lap splices between the column and its foundation, meet the requirements of Article 5.10.8.4.2a.

Column Collision

When the design choice is to redirect the collision load, follow the requirements given in Chapter 2 - Limit States and Loads. When the design choice is to provide structural resistance, design for the 600-kip equivalent static load as described in Article 3.6.5.

Design the column to withstand the collision force in shear only, not flexure, and do not consider the transfer of this force to the other elements such as bent caps, footings, piles, or drilled shafts. Use a 0.9 load factor for all dead loads and no live load for the axial compression used for shear design. The number of shear planes used in design is dependent on the bent configuration as described below:

◆ Bents with three or more columns that provide sufficient redundancy may use two shear planes to distribute the collision force.

◆ Bents with a lack of redundancy, such as two or fewer columns or large column spacing, such as straddle bents, require special consideration. For these, consider only one shear plane for the resistance of the collision force.

Use of a deflection wall between the columns is permitted, if necessary.

No further analysis is required for columns with a gross cross-sectional area no less than 40 sq. ft., a least dimension of no less than 5 ft. and column transverse reinforcement is composed of at least No. 4 ties at 12-in. maximum spacing or a No. 4 spiral at 9-in. maximum pitch.

Section 7 — Columns for Single Column Bents or Piers

Materials

Use TxDOT Class C concrete (f_c' = 3.6 ksi) and Grade 60 reinforcing steel. Higher concrete strengths may be used if needed in special cases.

Mass concrete may be needed depending on the size of the column.

Higher reinforcing steel grades may be used provided their use satisfies requirements in AASHTO.

Geometric Constraints

Consider using hollow pier sections where appropriate. Hollow piers subject to the column collision load must be protected from the collision consistent with the protection requirements elsewhere in this manual.

Structural Analysis

Account for second-order effects, with the structural model accounting for reduced stiffness from a cracked section.

Deflections from an analysis need to be consistent with boundary conditions of the actual structure.

Design Criteria

For columns subjected to bending under unfactored dead load, satisfy the minimum reinforcement requirements of Article 5.6.7, using an exposure factor consistent with the site and other bridge elements.

Detailing

For non-contact lap splices between the column and its foundation, meet the requirements of Article 5.10.8.4.2a.

Column Collision

When the design choice is to redirect the collision load, follow the requirements given in Chapter 2 - Limit States and Loads. When the design choice is to provide structural resistance, follow the requirements in Chapter 4 - Substructure Design, Sections 6 - Columns for Multi-Column Bents using one shear plane for the resistance of the collision force.

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 no less than 40 sq. ft., a least dimension of no less than 5 ft. and column transverse reinforcement is composed of at least No. 4 ties at 12-in. maximum spacing or a No. 4 spiral at 9-in. maximum pitch.

Section 8 — Post-Tensioned Concrete Bent Caps

Materials

Use TxDOT Class H concrete with a minimum f_c' = 5.0 ksi, and Grade 60 reinforcing steel. Higher steel grades may be used if needed in special cases.

Use 0.6-in. low-relaxation prestressing strand with a specified tensile strength, f_{pu}, of 270 ksi or high-strength steel bars meeting ASTM A722. All tendons and bars must be bonded.

Provide post tensioning system in accordance with Item 426, "Post-Tensioning," of the *TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges*.

Geometric Constraints

Criteria in this section are not intended for C-shaped bents or through-girder bents.

Cap depth cannot be less than cap width unless the cap is widened for the purposes of:

◆   meeting minimum support length per Article 4.7.4.4, or

◆   accommodating a cap-to-column connection when one or both elements are precast, or

◆   satisfying vertical clearance needs to a lower roadway.

See Section 4 for minimum cap widths, and column and drilled shaft dimensions based on beam or girder type.

Structural Analysis

Column-to-cap connection details must reflect assumptions of fixity made for post-tensioned cap design. Column stiffness and secondary force effects due to post-tensioning must be considered.

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

Round losses to 1 ksi. Determine prestress losses from elastic shortening, creep, shrinkage, and relaxation as prescribed for prestressed concrete in Chapter 3, Section 4, Pretensioned Concrete I Girders.

Use a minimum of 4 tendons in a cap.

Satisfy all stress limits for prestressing tendons and concrete as specified in Articles 5.9.2. Use limits for severe corrosive conditions in areas where de-icing agents are frequently used or where contact with salt water spray is possible.

Check limit states using the Strength I load combination and the Service I load combination for both tension and compression stress.

For multi-column bent caps, take design negative moments at the center line of the column. For hammerhead bents and multi-column bent caps with columns 4 ft. or wider, take design negative moments at the effective face of the column.

Minimize the number of stirrup spacing changes.

For typical multi-column bent caps supporting multiple beams, strut-and-tie modeling provisions of Articles 5.8 need not be considered. For bent caps supporting girders on high load multi rotational bearings or girders with large reaction forces that are defined as deep components according to Article 5.2, use the strut-and-tie design in accordance with Article 5.8.

Follow the provisions in Article 5.9.5 for Post-Tensioning Details.

Detailing

Use #5 stirrups, except as noted, 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. If torsional resistance is explicitly addressed in the design, ensure the stirrup detailing is consistent with AASHTO requirements.

Provide a minimum 5ft. tangent length of tendon from the anchorage head before introducing any curvature. Determine minimum radius of curvature of duct based on published values from suppliers for individual duct sizes.

Use minimum duct spacing according to Article 5.9.5.1.

Provide elevation and plan views showing the profile of centerline of ducts.

Show location and magnitude of final post-tensioning force(s) at critical section(s).

Provide the stressing and erection sequence on the plans, including form removal and girder placement. Bottom cap forms must stay in place until after first tendons are stressed.

Include the following information on the cap detail sheets:

◆　Assumed coefficient of friction and wobble coefficient for duct

◆　Assumed anchor set

◆　Assumed and maximum allowed eccentricity between duct and tendon

◆　Assumed long term losses

◆　Stressing and dead ends of tendon

◆　Jacking force = 0.77 x f_{pu} x A_{ps} = xx kips

Provide alternate reinforcing steel details where a known conflict between duct and typical reinforcing steel will occur. Include notes indicating all other adjustments to reinforcing steel must be made as directed by the Engineer of Record.

Include notes indicating that post-tensioning system/stressing sequence shop drawings must be submitted, reviewed, and approved by the Engineer of Record.

Section 9 — Lateral Restraint of Bridge Superstructures on Substructure

General

Lateral movement of superstructures can occur on water crossings due to flooding events and on grade separations due to cross slope with certain beam types. Provide effective lateral restraint in the form of shear keys as described in this section.

Bridges Crossing Water Features

Provide details for shear keys on abutment and bent caps of I-girder, U-beam, and spread box beam (X-beam) bridges that cross water features and meet any of the following criteria:

◆   River and stream crossings: the distance between the bottom of the beam and the 100 year high water level, as shown on the layout, is less than 4 feet

◆   Tidally influenced bridges: Refer to the *AASHTO Guide Specifications for Bridges Vulnerable to Coastal Storms* for guidelines.

I-Girder Bridges

Crossing Water Features:

Locate shear keys between the exterior and first interior beam on the upstream side of the bridge. This design practice supplements the use of dowels, if dowels are required.

U-Beam Bridges

Grade Separations:

Design U-beam bridges with shear keys on bent caps when the roadway has a single-direction cross-slope. Locate shear keys between the exterior and first interior beam on the high side of cross-slope.

Crossing Water Features:

Locate shear keys between the exterior and first interior beam on the upstream side of the bridge.

Spread Slab Beam and Spread Box Beam (X-beam) Bridges

Grade Separations:

Include shear keys on bent and abutment caps of X-beam and spread slab beam superstructures, for all allowable skews (0° through 30°) when the roadway has single direction cross-slope. Locate shear keys between the exterior and first interior beam on the high side of cross-slope.

Crossing Water Features:

Locate shear keys between the exterior and first interior beam on the upstream side of bridge.

Slab Beam, Box Beam, Decked Slab Beam, and Double-Tee Beam Bridges

Additional lateral restraint measures are not required for slab beam, box beam, decked slab beam, and double-tee beam structures, unless the bent cap does not have an earwall.

Steel Beam or Girder Bridges

Crossing Water Features:

Provide lateral restraints on steel bridges crossing water features.

Chapter 5 — Other Designs

Contents:

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 must conform to the current edition of the *American Association of State Highway and Transportation Officials (AASHTO) Manual for Bridge Evaluation* using Load Factor Rating unless otherwise directed by TxDOT. 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 Pretensioned Concrete I-Girders

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 for standard pad details. Note the bearings shown on standard drawing IGEB may not be applicable for skews over 60°.

Tapered bearings may be used if the taper does not exceed 0.050 ft./ft.

Use ¼-in. exterior pad layers. If using ¼-in. interior pad layers, disregard the requirements in Article 14.7.6.1, specifying exterior layers no thicker than 70% of internal layers.

Structural Analysis

Assume a temperature change of 70 °F after erection when calculating thermal movement in one direction (not total). Take T_{min} = 10 °F and T_{max} = 80 °F. For the panhandle region use T_{min} = 10 °F and T_{max} = 115 °F, for a total temperature change of 105 °F. Optionally, the temperature ranges in Article 3.12.2.2 may be used if appropriate.

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

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.

Ignore limit on $S_i^2/_n$ in Article 14.7.6.1.

Design Criteria

Follow Design Method A in 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 Article 14.7.6.3.5.

Rotation is acceptable if the total compressive deflection equals or exceeds $\dfrac{\theta(0.8L)}{2}$, where L
is the pad length defined in *AASHTO LRFD Bridge Design Specifications*, and θ is the total
rotation. Estimate compressive deflection using Figure C14.7.6.3.3-1.

◆   Calculate total rotation for dead and live load plus 0.005 radians for construction uncertainties
as required by Article 14.4.2.1. Take maximum

live load rotation as $\dfrac{4\Delta}{SpanLength}$, 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 (in.)

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

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

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

◆   Use h_{rt}, instead of total pad height when checking stability as required in Article 14.7.6.3.6.

Detailing

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

Section 3 — Strut-and-Tie Method

Structural Analysis

Do not use Strut and Tie Modeling for standard girders and bent caps. Strut and Tie Modeling is appropriate to use when designing footings, dapped beam ends, post-tensioning anchorage zones, deviation diaphragms, bents that use high load bearings, and other special designs.

Design Criteria

For members designed with strut-and-tie, check the calculated shear, using the Service I Load Combination, against the resistance from Equation C5.8.2.2-1.

Section 4 — Corrosion Protection

Guidance

In areas of the state where de-icing agents 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 on TxDOT's web page for statewide and district specific recommendations. Consult the Bridge Division for recommended corrosion protection measures in marine environments.

Section 5 — Concrete Culverts

This section applies to cast-in-place and precast concrete box culverts, inlets, junction boxes, and manholes.

Materials

For cast-in-place concrete culverts, use Class C concrete (f_c' = 3.6 *ksi*) with the following exceptions: use Class S concrete (f_c' = 4.0 *ksi*) for top slabs of culverts with overlay, with 1-to-2 course surface treatments, or with the top slab as the final riding surface (i.e., "direct traffic"). Refer to district-specific corrosion protection requirements when designing "direct traffic" culverts for regions where bridge decks are exposed to de-icing agents and/or saltwater spray with regularity. If thus required, use Class S (HPC) concrete for the top slab.

For precast concrete culverts, use Class H concrete (f_c' = 5.0 *ksi*).

Use Grade 60 reinforcing steel or deformed welded wire reinforcement (WWR) meeting the requirements of ASTM A1064. Refer to district-specific corrosion protection requirements when designing "direct traffic" culverts for regions where bridge decks are exposed to de-icing agents and/or saltwater spray with regularity. If corrosion resistant reinforcement is required, Grade 60 hot-dip galvanized reinforcing steel may be used in accordance with Item 440.

Geometric Constraints

- The maximum skew angle for box culverts is 45°.
- Provide 1.5 in. of clear cover for reinforcement in cast-in-place concrete culverts.
- Provide 1.0 in. of clear cover for reinforcement in precast concrete culverts.
- Provide 2.0 in. of clear cover above the top layer of reinforcement in the top slab of "direct traffic" concrete culverts.

Structural Analysis

- Analyze each box culvert as a simply supported two-dimensional frame. Do not apply the edge beam requirement in Article 12.11.2.1.
- For culverts with less than two feet of fill (i.e., "direct traffic"), distribute the weight of curbs and railing over the entire top slab width if the slab is no wider than 32 ft. Otherwise, distribute the curb and railing loads over 16 ft.

Design Criteria

- Do not apply the provisions for design tandem as described in Article 3.6.1.3.1.

◆ Do not apply the variable axle spacing described in Article 3.6.1.2.2. Set the spacing between the two 32.0-kip axles equal to 14.0 ft.

◆ Unless site-specific information is available, assume cohesionless soil with unit weight equal to 120 pcf and friction angle equal to 30°. Use Rankine's active earth pressure coefficient to compute lateral earth pressure. Precast box culverts may be designed for the earth loads defined in ASTM C1577.

◆ Do not apply the water loads indicated in Table 3.4.1-1 for structural analyses of concrete culverts, unless warranted by site conditions.

◆ Limit the equivalent height of soil, h_{eq}, from Table 3.11.6.4.1 to a maximum value of 4.0 ft.

Detailing

◆ Place reinforcement for skewed ends as directed in the SCC-MD, SCP-MD, or MC-MD standard sheets.

Chapter 6 — Archiving Design Notes

Contents:

Section 1 — Overview

Introduction

In order to comply with Federal Highway Administration (FHWA) requirements for maintaining records, the Bridge Division implemented a procedure for archiving bridge design notes in TxDOT's bridge inspection database management system. For all designs, perform the procedures contained in this chapter when a design is complete.

Section 2 — Procedure for Archiving Design Notes

Scan Notes

Scan the notes (or convert electronic files) and gather them into a single PDF file. Create separate PDF files for each bridge. In the case of a single design done for twin structures, submit the same notes under two separate NBI numbers.

A hard copy of computer software input files is not required. Hard copies of computer software output should be included with the following:

◆ Input echo

◆ Key pages of output, annotated by hand if necessary, such that the design's outcome, including controlling load case and limit state, can be understood by review of the output annotation.

◆ Key pages of output, annotated by hand if necessary demonstrating that other load cases and limit states do not control the outcome of the design.

Include the following required design elements in the PDF file:

◆ Calculations and other documentation establishing the bridge's superstructure design satisfies controlling load cases and limit states, for the following elements:
- Girders or beams
- Stringers
- Floor beams
- Trusses, including secondary elements such as bracing and gusset plates
- Arches and hangers, including secondary elements such as bracing and gusset plates
- Cable stays
- Other elements not specifically excluded

◆ Calculations and other documentation establishing the bridge's substructure design satisfies controlling load cases and limit states, for the following elements:
- Cap beams
- Columns, Towers, and Pylons
- Other elements not specifically excluded

◆ Calculations and other documentation establishing the bridge's foundation design satisfies design requirements, for the following elements:
- Piling
- Drilled shafts
- Spread footings

- Other elements not specifically excluded

Additionally, the file should contain:

◆ Completed TxDOT Quality Control Cover Sheet from the Quality Control and Quality Assurance Guide at http://ftp.dot.state.tx.us/pub/txdot-info/library/pubs/bus/bridge/qa_qc_guide.pdf. Consultant Designers are allowed to use their own cover sheet if it is similar to the TxDOT Quality Control Cover Sheet.

◆ Bridge layout at the time of the original design

◆ Communication directly related to the included elements

◆ Do not include bridge geometry runs (BGS, Geomath, spreadsheets, etc.).

Naming Convention

Name the file using the following naming convention:

◆ Design notes: NBI_DN_yyyy-mm, with yyyy-mm being the year and month the PDF file is submitted (ex. 1234567890abcde_DN_2015-01)

◆ Change Orders: NBI_CO_yyyy-mm (ex. 1234567890abcde_CO_2015-01)

Delivering the File

Deliver the PDF file using DropBox, at https://ftp.dot.state.tx.us/dropbox/

◆ Select Drop-off.

◆ Enter Sender Information.

◆ Enter Recipient Information (use email: BRG_Design_Notes@txdot.gov)

◆ Attach the design notes PDF file.

◆ Select Drop-off the File(s).

If the PDF is created by a Consultant designer, the Consultant sends the PDF bridge design notes to the TxDOT contract manager or the District Bridge Engineer, who will submit the notes at the DropBox link above.

Once the file has been uploaded, the auto-generated email will serve as notification to the Bridge Management Group that files have been received. The Bridge Management Group is responsible for transferring the files into TxDOT's bridge inspection database management system, where they will become part of the permanent bridge file.