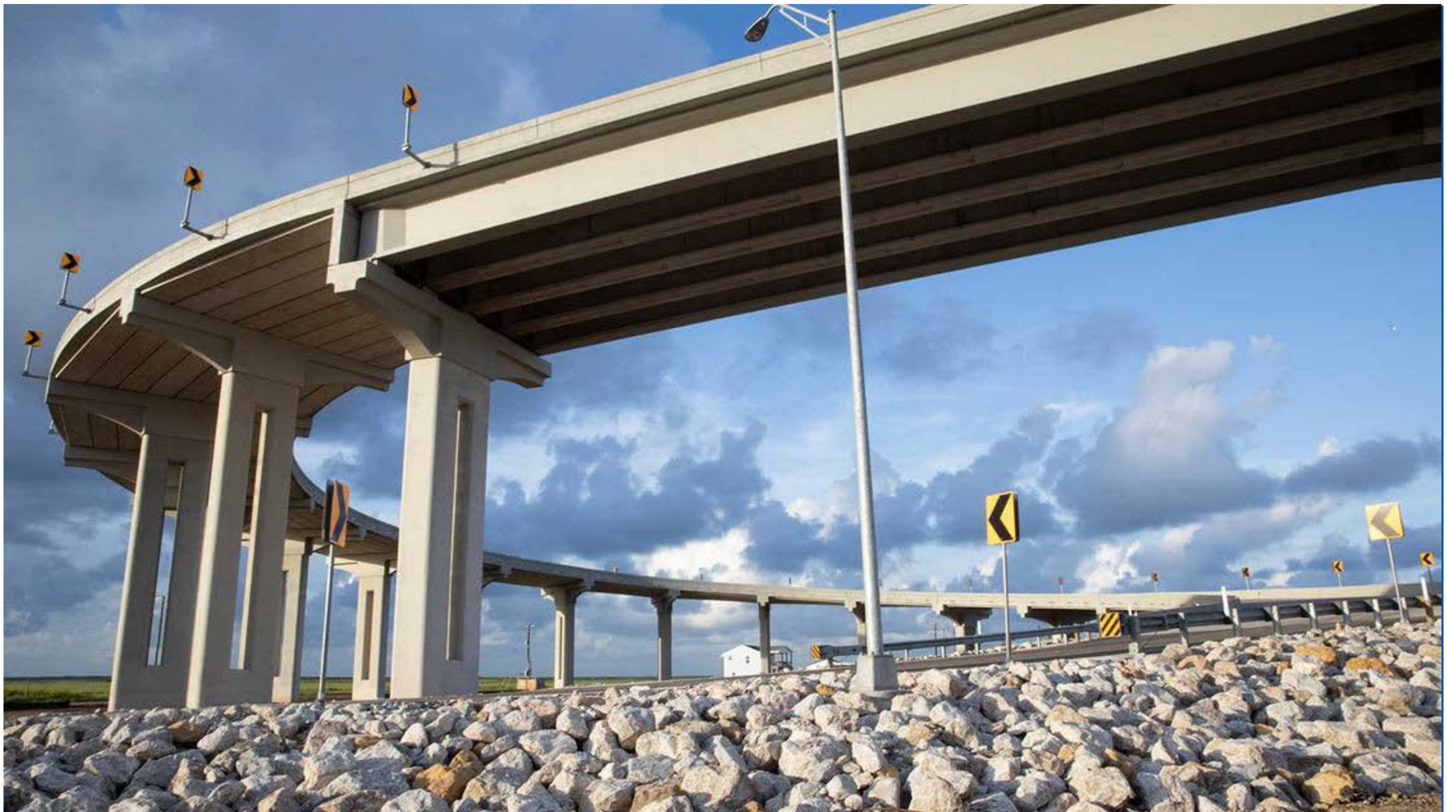


Texas Department of Transportation

Bridge and Structures Design Manual

July 2026

Manual Notice 2026-1

From: Jamie Farris, P.E., Director, Bridge Division

Manual: *Bridge and Structures Design Manual*

Effective Date: July 29, 2026

Purpose

The TxDOT *Bridge and Structures Design Manual* documents the policy on bridge, culvert and inlet, and ancillary structure design in Texas. It assists Texas bridge designers in applying provisions documented in the AASHTO LRFD Bridge Design Specifications, to which designers should adhere unless directed otherwise by this document.

Changes

This manual is a new document, as it merges the *Bridge Design Manual-LRFD* and the *Bridge Design Guide*. It contains policy language as well as commentary language to provide guidance and background information.

Updates consist of the following: reorganized the contents, added 3D Modeling requirements (Section 2.2), updated post letting alternate designs requirements (Section 2.6), added bridge geometry requirements (Section 4.2 and the various beam types in Chapter 5), added curved bar nodes (Section 4.13), reorganized pretensioned beam design requirements (Section 5.3), added requirement of diaphragms for spliced girders (5.4.1), updated segmental requirements (Section 5.4.2), revised steel beam bracing and split-pipe stiffener requirements (Section 5.6), added steel trapezoidal box girders (Section 5.6.3), reorganized substructure section (Chapter 6), added footing requirements (Section 6.4), added ancillary structures requirements (Chapter 8), revised the U beam design example to be an X beam design example (Appendix B), and added Lean-on Bracing Guide (Appendix D).

Supersedes

This revision supersedes version 2024-1 (September 2024) of the *Bridge Design Manual – LRFD* and version 2023-1 (January 2023) of the *Bridge Design Guide*.

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.

Contents

Chapter 1 About this Manual

1.1 Purpose

American Association of State Highway and Transportation Officials (AASHTO) defines Load and Resistance Factor Design (LRFD) as a design methodology that makes use of load and resistance factors based on the known variability of applied loads and material properties. 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.

The purpose of this manual is to document policy and accompanying commentary for the design of structures, including bridges, culverts, and ancillary structures, in the state of Texas.

It assists Texas bridge designers in applying provisions documented in the *AASHTO LRFD Bridge Design Specifications, 10th Edition, 2024*, which designers should adhere to unless directed otherwise by this document.

All <bracketed> references in this manual are from the *10th Edition of AASHTO LRFD Bridge Design Specifications*, unless noted otherwise.

All Item numbers (i.e. Item 440) references in this manual are to the *Texas Department of Transportation Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges,* unless noted otherwise.

> 3D Bridge Modeling Best Practices. Texas Department of Transportation. https://www.txdot.gov/content/dam/docs/division/brg/3d-bridge-modelling-best-practices.pdf

Bridge Detailing Guide. Texas Department of Transportation. https://ftp.txdot.gov/pub/txdot-info/brg/design/bridge-detailing-guide.pdf

Bridge Inspection Manual. Texas Department of Transportation. https://www.txdot.gov/manuals/brg/ins/index.html

Bent Protection Guide. Texas Department of Transportation. https://ftp.txdot.gov/pub/txdot-info/brg/design/bent-pier-protection-guide.pdf

Bridge Project Development Manual. Texas Department of Transportation. https://www.txdot.gov/manuals/brg/bpd/index.html

Bridge Railing Manual. Texas Department of Transportation. https://www.txdot.gov/manuals/brg/rlg/index.html

BridgeLink PGSuper Design Guide. Texas Department of Transportation. https://ftp.txdot.gov/pub/txdot-info/isd/txdotapps/support/design_guidelines_pgsuper.pdf

Geotechnical Manual-LRFD. Texas Department of Transportation. https://www.txdot.gov/manuals/brg/geo_lrfd/index.html

Guidelines for the Use of Steel Piling for Bridge Foundations. Texas Department of Transportation. https://ftp.txdot.gov/pub/txdot-info/brg/geotechnical/steel-pilings.pdf

Hydraulic Design Manual. Texas Department of Transportation. https://www.txdot.gov/manuals/des/hyd/index.html

Quality Control and Quality Assurance Guide. Texas Department of Transportation. https://ftp.txdot.gov/pub/txdot-info/library/pubs/bus/bridge/qa_qc_guide.pdf

Roadway Design Manual. Texas Department of Transportation. https://www.txdot.gov/manuals/des/rdw/index.html

Preferred Practices for Steel Bridge Design, Fabrication, and Erection. Texas Department of Transportation. https://ftp.dot.state.tx.us/pub/txdot-info/library/pubs/bus/bridge/steel_bridge.pdf

Recommended Span Lengths for Prestressed Concrete Beams. Texas Department of Transportation. https://www.txdot.gov/content/dam/docs/division/brg/prestressed-concrete-beam-span-lengths.pdf

Scour Analysis Guide. Texas Department of Transportation. https://ftp.txdot.gov/pub/txdot-info/des/guides/scour-guide.pdf

Scour Evaluation Guide. Texas Department of Transportation. https://ftp.txdot.gov/pub/txdot-info/library/pubs/bus/bridge/scour-guide.pdf

Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges. Texas Department of Transportation. https://www.txdot.gov/content/dam/docs/specifications/2024/spec-book-0924.pdf

Structure Design - Corrosion Protection Guide. Texas Department of Transportation. https://ftp.txdot.gov/pub/txdot-info/brg/design/corrosion-protection-guide.pdf

Links to websites:

Bridge Publications website. Texas Department of Transportation. https://www.txdot.gov/business/resources/highway/bridge/bridge-publications.html

Bridge Standards website. Texas Department of Transportation. http://www.dot.state.tx.us/insdtdot/orgchart/cmd/cserve/standard/bridge-e.htm

Digital Delivery Website. Texas Department of Transportation. https://www.txdot.gov/business/resources/digital-delivery.html

Extended Span Precast Girders website. Texas Department of Transportation. https://www.txdot.gov/business/resources/highway/bridge/extended-span-precast-girders.html

Contains:
Long Span Precast I-Girder Sections and
Long Span Precast U-Girder Straight and Curved Sections.

Statewide Planning Map website. Texas Department of Transportation https://www.dot.state.tx.us/apps/statewide_mapping/StatewidePlanningMap.html

TxDOT and FHWA Engineering Software website. Texas Department of Transportation. https://www.txdot.gov/business/resources/design-tools-training/txdot-fhwa-engineering-software.html

1.2 Organization

The information in this manual is organized as follows:

Table 1.1: Manual Organization

Chapter	Chapter Name	Description
1	About this Manual	Introductory information on the purpose and organization of the manual.
2	Bridge Design Project Requirements	Information on 3-Dimensional (3D) bridge modeling, load rating new bridges, archiving design notes, and quality control and assurance.
3	Limit States and Loads	General information on limit states and load factors.
4	General Requirements	Design guidelines for various miscellaneous design aspects including bridge geometry, width, vertical clearance, widenings, steel-reinforced elastomeric bearings, strut-and-tie method, corrosion protection, non-contact lap splices, and vehicle collision.
5	Superstructure Design	Policy on design of specific bridge superstructure components.
6	Substructure Design	Policy on design of specific bridge substructure components.
7	Concrete Culverts	Policy on design of concrete culverts.
8	Ancillary Structures	Policy on structural design of ancillary structures, including sign supports, high mast illumination poles, roadway illumination poles, and traffic signal poles.

This document integrates both policy and accompanying commentary. Commentary sections are clearly identified by a header titled Commentary and in a shaded box. It is important to note that commentary is not considered policy. All content that is not explicitly marked as commentary constitutes the policy.

Commentary

This document combines two previously separate documents: the Bridge Design Manual-LRFD and the Bridge Design Guide.

1.3 Feedback

Direct any questions or comments on the content of the manual to the Bridge Division, Texas Department of Transportation (TxDOT) (BRG_Design@txdot.gov).

Chapter 2 Bridge Design Project Requirements

2.1 Overview

This chapter documents policy for bridges and other structures that is beyond the design of the bridge or structure itself.

2.2 3D Bridge Modeling

Create a 3D bridge model using OpenBridge Modeler (OBM) for all projects that utilize OpenRoads Designer (ORD) for roadways, except for bridge class culverts, rail retrofits, rehabilitations, and repair projects.

- Use 3D elements for slab, beams, bearing pads, bearing seats, abutments, wingwalls, bridge rails, caps, columns, and foundations. Convey information in the model to the best of the program's capabilities. If the program is unable to accurately model the structure, send an email describing the situation to bridge3ddesign@txdot.gov.
- Use the OBM version indicated on the TxDOT workspace website at the time that the ProjectWise project folder is created, or when the consultant contract is executed. The use of subsequent versions may be used at the district's discretion when the ProjectWise project folder is compatible with the software version.
- Submit the 3D bridge model at Preliminary Bridge Layout Review (PBLR) and 100% submittals with all reference files used in the bridge model.
- Prepare a letter report which includes the findings of the comparison of OBM 3D bridge model geometry and quantities compared to traditional design methods. Include a detailed discussion of the differences between methods and recommended enhancements in OBM software.
- The contract deliverable for construction will be the plan sheets with 2-Dimensional (2D) details, unless specified otherwise by the district.

- The TxDOT Project Manager (PM) will upload the 3D bridge model to the ProjectWise project folder.

Commentary

TxDOT is transitioning toward submitting the 3D bridge model with the contract plans for contractor's information only. In preparation for this shift, all users should stay up to date on the current digital delivery program requirements. Refer to *TxDOT's Digital Delivery* website for the most recent guidance. Guidance for 3D bridge modeling is found in the *TxDOT 3D Bridge Modeling Best Practices* document.

2.3 Load Rating New Bridges

2.3.1 Purpose

FHWA's Metrics for the Oversight of the National Bridge Inspection Program requires all bridges to be rated for their safe load carrying capacity for current conditions in accordance with the *AASHTO Manual for Bridge Evaluation (MBE).* Load rate for all state legal vehicles and routine permit loads. Provide load ratings in the design plans of all new bridges as described herein.

Commentary

The MBE provides detailed guidelines and procedures for evaluating the load carrying capacity of bridges, ensuring compliance with the *National Bridge Inspection Standards* (NBIS).

2.3.2 Load Rating Requirements

Load rate in accordance with the current AA*SHTO Manual for Bridge Evaluation* (MBE) with current interims. Use the Load and Resistance Factor Rating (LRFR) method outlined in Chapter 6 Part A of the MBE.

Use HL-93 live load as described in <Article 3.6.1.2> unless design for a special vehicle is specified or warranted. Use the fatigue loading as described in <Article 3.6.1.4> for fatigue limit states.

Use the limit states and associated load factors as indicated in Table 2.3.2-1 below in lieu of MBE Table 6A.4.2.2-1.

For new construction, use a condition factor of 1.0. Do not apply a system factor. If warranted, include any adjustment for ductility, redundancy, or operational classification as load modifiers on the force effects within the design.

For prestressed and post-tensioned concrete bridges at the service flexure limit state, assume a maximum allowable tensile stress of 0.19 $\sqrt{f_c'}$, regardless of exposure environment.

The following elements require load ratings:

- Prestressed beams and girders for service flexure, ultimate flexure, and ultimate shear.
- Steel girders (straight and curved, I-shaped and tubs) for service flexure, ultimate flexure, and ultimate shear. Non-redundant steel tension members (NSTMs) require a fatigue load rating.
- Post-tensioned concrete superstructures for service flexure, ultimate flexure, and ultimate shear.
- Reinforced concrete superstructures for ultimate flexure and ultimate shear.

Table 2.3.2-1: TxDOT Limit States and Load Factors for Load Rating New Designs (Replaces MBE Table 6A.4.2.2-1)

Bridge Type	Limit State	Dead Load (1)		Design Live Load	
				Inventory	Operating
		γ_{DC}	γ_{DW}	γ_{LL}	γ_{LL}
Prestressed Concrete	Strength I	1.25	1.50	1.75	1.35
	Service III	1.00	1.00	0.80	N/A
Structural Steel	Strength I	1.25	1.50	1.75	1.35
	Service II	1.00	1.00	1.30	1.00
	Fatigue (2)	0.00	0.00	0.80	N/A
Reinforced Concrete (3)	Strength I	1.25	1.50	1.75	1.35
Post Tensioned Concrete (4)	Strength I	1.25 max, 0.9 min	1.50 max, 0.65 min	1.75	1.35
	Service I (5)	1.00	1.00	1.00	N/A
	Service III	1.00	1.00	0.80	N/A

1) For most new structures, no dead load for wearing surface is employed in design and accounting for added future wearing surface is not required. Also, note that the entire concrete deck thickness is to be included in the DC load effects and the resisting capacity (include precast panels, no sacrificial thickness assumed).
2) Only evaluate the fatigue limit state for non-redundant tension steel members (NSTMs).
3) Superstructure (e.g. non-standard reinforced concrete slab or slab and girder (pan form spans)) and substructure (interior bent caps). Do not load rate bridge decks supported by girders, except where noted.
4) Include the effects of creep, shrinkage, secondary effects (due to prestress/post-tensioning), uniform temperature change, and temperature gradient as indicated in Table 2.3.2-2.
5) Applies to post-tensioned straddle bents and transverse analysis of post-tensioned concrete bridge decks.

Table 2.3.2-2: Additional Loads and Load Factors for Post-Tensioned Concrete (6)

Limit State	CR, SH	PS	TU (7)	TG
Strength I	1.25 max, 0.9 min (8)	1.0	1.0	N/A
Service I	1.0	1.0	1.0	0.50
Service III	1.0	1.0	1.0	0.50

6) Adapted from <Table 3.4.1-1>, <Table 3.4.1-3>, and <Table 3.4.1-6>.
7) Only applicable in framed structures.
8) Applies to segmental superstructures. For non-segmental superstructures use 1.0.

The following elements generally do not require load ratings, unless specifically needed based on the request of TxDOT or where the substructure might be a controlling element:

- Reinforced concrete bent caps for ultimate flexure and ultimate shear.
- Reinforced concrete inverted-tee bent caps for ultimate flexure, ultimate shear, and ledge and hanger checks.
- Post-tensioned bent caps for service flexure, ultimate flexure, and ultimate shear.
- Steel bent caps (e.g. integral caps, straddle box caps, etc.) for service flexural, ultimate flexure, ultimate shear, and fatigue.

Do not load rate bridge decks with the exceptions of transversely post-tensioned decks of segmental box girders and bridge decks with girders spaced wider than 12 feet.

Determine the controlling load rating based on the range of limit states indicated in Table 2.3.2-1, identify the location (e.g. girder, span, position) both in the supporting design calculations and the *New Bridge Load Rating Summary for Design Load* (Form 2731), as well as on the bridge layout.

Contact the Bridge Division (BRG_Design@txdot.gov) for guidance on unique structures or elements not addressed above.

2.3.3 Standard Bridges

For bridges developed from standards for superstructure, use the load rating information documented on the standard. If there is no load rating information in the bridge standards, assume an inventory rating factor of 1.0 and operating rating factor of 1.3, which are lower bound numbers. Standards modified for aspects that would not otherwise affect the load rating, may also utilize the load rating information documented on the standard. No supporting calculations are needed for such standard and modified standard bridges, but load rating information must be transcribed to the *New Bridge Load Rating Summary for Design Load* (Form 2731) and indicated on the bridge layout.

2.3.4 Documentation of Load Rating

Provide the load rating calculations and the *New Bridge Load Rating Summary for Design Load* (Form 2731) with the archived bridge design notes that are submitted as outlined in Section 2.4. Submit the load ratings based on the component, limit state, and controlling location as indicated on Form 2731. Include the final controlling HL-93 Rating Factor (RF) at the Inventory (Inv) and Operating (Opr) Levels. Include the controlling inventory and operating load ratings on the bridge layout as indicated below and in the *TxDOT Bridge Detailing Guide*.

New construction:

HL-93 Loading	Superstructure Inv/Opr Ratings= X.XX/Y.YY
	Substructure Inv/Opr Ratings= X.XX/Y.YY or “Substructure Not Rated”

2.4 Archiving Design Notes

2.4.1 General

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 electronic *Bridge Inspection Management System*. These procedures must be followed upon completion of all bridge designs and updated based on any requests for information or change orders that occur during construction.

2.4.2 Design Notes

Scan the notes (or convert electronic files) and gather them into a single Portable Document Format (PDF) file. Create separate PDF files for each bridge. In the case of a single design done for twin structures, provide a copy of the design notes for each unique National Bridge Inventory (NBI) number titled as described in Section 2.4.3.

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

- Input echo.
- Key pages of output, annotated 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 if necessary, demonstrating that other load cases and limit states do not control the outcome of the design.

Include the following design elements in the PDF file, as applicable:

- Calculations, design assumptions, and other documentation establishing that 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, design assumptions, and other documentation establishing that 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, design assumptions, and other documentation establishing that the bridge's foundation design satisfies design requirements, for the following elements:
  - Piling.
  - Drilled shafts.
  - Spread footings.
  - Other elements not specifically excluded.
- Calculations, design assumptions, and other documentation establishing the bridge's load rating. Include Form 2731.
- Completed TxDOT Quality Control Cover Sheet from the *TxDOT Quality Control and Quality Assurance Guide*. Consultant Designers are allowed to use their own cover sheet if it contains the same information as 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 calculations and any related files.

2.4.3 Naming Convention

Name the file using the following naming convention:

- Design notes: DD-CCC-CCCC-SS-SSS_DN_YYYY-MM.

    DD = 2-digit District ID.

    CCC = 3-digit County ID.

    CCCC = 4-digit Control.

    SS = 2-digit Section.

    SSS = 3-digit Structure Number.

    DN = Design Notes.

    YYYY-MM being year and month the PDF file is submitted.

    ex. 12-345-6789-0a-bcd_DN_2015-01.

- Change Orders: DD-CCC-CCCC-SS-SSS_DN_CO_YYYY-MM.

    DD = 2-digit District ID.

    CCC = 3-digit County ID.

    CCCC = 4-digit Control.

    SS = 2-digit Section.

    SSS = 3-digit Structure Number.

    DN = Design Notes.

    CO = Change Order.

    YYYY-MM being year and month the PDF file is submitted.

    ex. 12-345-6789-0a-bcd_DN_CO_2015-01.

2.4.4 Electronic Submittal of Bridge Design Notes

Treat bridge design notes as confidential documents and transmitted using secure, encrypted methods. Use Box.com or an equivalent secure file-sharing service. Do not transmit design notes via unencrypted channels, such as standard email attachments.

Prior to transmitting bridge design notes electronically, notify the recipient via email. In the notification, indicate that a secure electronic submittal containing bridge design documentation will be delivered.

The responsibility for submitting bridge design notes depends on the origin of the document. If the PDF is prepared by a consultant, it must be submitted to the TxDOT Contract Manager or the District Bridge Engineer. If the PDF is prepared by TxDOT staff, it must be submitted directly to the District Bridge Engineer.

The District Bridge Engineer is responsible for transferring the files into TxDOT's electronic *Bridge Inspection Management System*, where the bridge design notes become part of the permanent bridge file.

2.5 Quality Control and Quality Assurance (QC/QA)

2.5.1 General

FHWA requires that a Quality Assurance Program, which includes quality control procedures, be in place and followed. The Quality Control and Quality Assurance (QC/QA) process applies to

- all types of bridge and structure projects done for TxDOT regardless of who designs it (Bridge, District, or consultant).
- all calculations and plan sets.
- the development of design guidelines, design examples, spreadsheets, standard details, manuals, and specifications.

At a minimum, meet the guidance provided in the *TxDOT Quality Control and Quality Assurance Guide*. Alternate processes that meet these minimum requirements are allowed.

Include QC/QA forms in every design note package. Fill the form in completely. Fill out title blocks on all bridge plan sheets, including layouts, completely and show the designer and checker initials.

In addition to the basic QC/QA process covered in the *TxDOT Quality Control and Quality Assurance Guide*, independent analysis/analyses may be required for complex or exotic structures or structural elements.

2.5.2 Consultant Prepared

For consultant-prepared plans, the consultants are required to submit their Quality Control Plan in writing prior to starting work or as otherwise directed in the contract. TxDOT reserves the right to review and approve the consultants' quality control process.

2.5.3 Independent Analysis

Independent analysis/analyses may be required for complex or exotic structures or structural elements. The independent analysis must meet the following requirements:

- Completed by a Professional Engineer licensed in the State of Texas who is precertified for the applicable bridge design categories by TxDOT.
- Meet the requirements of this document.
- Be conducted without the aid of the original design calculations.
- Use structural engineering design/analysis software different from what was used for the original design, when available.
- Determine that the original plans as designed by the Engineer of Record are in compliance with established design criteria.
- Generate a separate set of design calculations in a report that documents any changes or recommendations regarding the original plans.

The independent analysis report will then be compared to the original design by the Engineer of Record (EOR). This comparison must be documented and any issues resolved prior to construction.

2.6 Alternate Designs

2.6.1 General

The allowance for post-letting alternatives must be explicitly stated in the project general notes and/or on plan sheets. Consideration should be given to providing allowance in both locations.

Acceptance or denial of an alternate is at the sole discretion of TxDOT. All alternate designs must be signed and sealed by a Professional Engineer licensed in the State of Texas who is precertified for the applicable design categories by TxDOT. All costs and time impacts associated with Contractor proposed alternates are borne solely by the Contractor, including but not limited to plan development, coordination, submission, redesign, analyses, and shop plan reviews.

The Contractor must submit the alternate design for review and approval. Submit complete design calculations, plans, and documentation demonstrating compliance with TxDOT standards and the original design intent.

Allowing precast alternates is only intended for individual components, such as the exterior beam or bent cap. Not for entire bridge redesigns that alter span depth or arrangement. Post-letting do not develop entire bridge redesigns, unless specific plan sheets clearly state entire bridge redesign is permitted. In such cases, the contractor has the option to propose a new design, assuming the role as the new Engineer of Record, and the new plans must adhere to the design intent of the original plans. The new Engineer of Record must be a Professional Engineer licensed in the State of Texas who is precertified for the applicable design categories by TxDOT and provide signed and sealed plans and calculations for the redesign.

For water crossings, the Contractor must submit hydraulic, scour, and geotechnical design calculations for alternates that change the superstructure depth; span arrangement; substructure size; and/or column and footing size and spacing, as well as the design calculation for the bridge components. Any alternatives that change the hydraulic opening and characteristics require hydraulic and scour redesign.

For grade separations, the Contractor must submit vertical clearance calculations for alternates that change the superstructure depth. Minimum vertical clearance as shown in the plan must be met or exceeded. Alternate span arrangement will not be accepted.

2.6.2 Concrete Alternatives

If it is desired to allow post-letting precast and prefabricated alternatives to the as designed cast-in-place concrete or precast concrete bridge superstructures or substructures, include the precast alternates working drawing(s) found on the TxDOT Bridge Standards website. These working drawings must be modified on a project-by-project basis and then signed and sealed by a Professional Engineer licensed in the State of Texas who is precertified for the applicable bridge design categories by TxDOT.

Develop alternates using the TxDOT Bridge Standards and working drawings for precast elements. For alternates not covered by the standards, develop these alternates based on the relevant concepts demonstrated within the standards and working drawings.

Do not develop complex precast alternatives that will require additional research or investigation without prior approval from Bridge Division.

2.6.3 Steel Alternates

Use the TxDOT Bridge Standards and the *TxDOT Preferred Practices for Steel Bridge Design, Fabrication, and Erection* as the basis for developing steel alternates.

Bridge Division approval is required for:

- Non-standard configurations.
- Non-redundant tension steel member classification changes.
- Use of alternate paint systems or bolt types.

Post-letting do not develop entire bridge redesigns, unless specific plan sheets clearly state entire bridge redesign is permitted. In such cases, the contractor has the option to propose a new design, assuming the role as the new Engineer of Record, and the new plans must adhere to the design intent of the original plans. The new Engineer of Record must be a Professional Engineer licensed in the State of Texas who is precertified for the applicable bridge design categories by TxDOT and provide signed and sealed plans and calculations for the redesign.

2.6.4 Bent Caps and Abutments

Provide equal or better corrosion resistance to the originally designed bent cap and cap to column connection as shown in the project plans.

Precast, prestressed bent caps that are designed to “no tension” using uncoated strand and uncoated reinforcement provides an equal corrosion resistance to uncoated, epoxy coated, and galvanized reinforcement. The cap to column connection must meet the corrosion protection level originally shown in the plans.

If the precast bent cap alternate reduces the number of columns, design the alternate bent cap and columns to meet the requirements for vehicular collision presented in Section 3.8.2 and Section 4.14.

Do not use the following without prior approval from TxDOT Bridge Division:

- Hollow shells subsequently infilled.
- Hollow voids.
- Lightweight concrete.
- Separate bent cap pieces, abutment cap pieces, backwalls, or wingwalls joined together in the field.

2.6.5 Precast Concrete Columns

Contact the Bridge Division (BRG_Design@txdot.gov) for approval prior to using precast columns.

If approved, design precast column to meet the same vehicular, railroad, or vessel collision loads originally designed for in the plans.

2.6.6 Prefabricated Steel Bent Caps

Design prefabricated steel bent caps for load path redundancy, system redundancy, or internal redundancy. Due to level of effort associated with pursuing acceptance of system redundancy or internal redundancy, obtain prior approval from the TxDOT Bridge Division before progressing with alternate design.

Chapter 3 Limit States and Loads

3.1 Overview

Load and Resistance Factor Design (LFRD) is a methodology that makes use of load factors and resistance factors based on the known variability of applied loads and material properties.

3.2 Importance Factor

Classify all bridge designs as typical bridges, as defined in <Article 1.3.5>, when applying the operational importance factor, η_I, to strength limit states.

3.3 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. Do not classify single-cell boxes and single-column bents nonredundant, unless approved by TxDOT Bridge Division.

3.4 Foundations

Determine design loads and combinations for bridges as described by *TxDOT's Geotechnical Manual – LRFD*.

For foundation loads on typical multi-column bents and abutments, distribute the live load equally to all supporting foundations, assuming all lanes are loaded. Use the multiple presence factor, m, per <Article 3.6.1.1.2>.

3.5 Dead Loads

Do not design for future wearing surfaces unless specifically directed by the District. Only apply dead load of wearing surfaces when such a wearing surface is part of the current design or a known future application.

The engineer may reduce the maximum load factor for wearing surfaces and utilities (DW) in <Table 3.4.1-2> to 1.25.

Commentary

Examples of when overlay should be considered in the design includes but are not limited to:

- Bridge widenings where overlay is added to match existing conditions or overlay is being replaced.
- Accelerated bridge construction that might utilize overlay to reconcile beam camber to a finished profile grade or overlay contingency for final profile.
- Planned overlays such as polyester polymer concrete (PPC) to be applied before the bridge is opened as part of a preservation or preventative maintenance plan.

3.6 Live Loads

3.6.1 Live Load

Use design vehicular live load, designated as HL-93, as described in <Article 3.6.1.2> unless design for a special vehicle is specified or warranted.

Design widenings for existing structures using HL-93.

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>.

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>.

Do not apply the provisions of <Article 3.6.1.1.2> to the extreme limit state when evaluating system redundancy as specified in Section 5.6.3.

3.6.2 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.

3.6.3 Live Load Deflection

Check live load deflection using <Article 2.5.2.6.2> and <Article 3.6.1.3.2>. Calculate deflection using a live load distribution factor equal to the number of lanes divided by the number of girders. Use the deflection limits given in <Article 2.5.2.6.2>.

3.6.4 Pedestrian

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

3.7 Wind Load

3.7.1 Wind on Segmental Structures During Construction

Use the following wind speeds for load combinations defined in <Article 5.12.5.3.3>:

- *Load Cases c and d* – Use wind speeds from Service IV.
- *Load Cases e and f* – Use wind speeds from Service I.

Use the following wind speeds for load combinations defined in <Article 5.12.5.3.4a>:

- *Load Cases Strength III* – Use wind speeds from Strength III multiplied by 0.95.
- *Load Cases Strength V* – Use 75 mph.

3.8 Extreme Events

3.8.1 Extreme Event Limit State

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 or 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 floe, 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 designed for except in regions near Big Bend as noted in the subsequent section on Earthquake Effects.
>
> Provisions under Extreme Event II must be designed for only when vehicular collision or vessel collision evaluation is required. For dead load (DC and DW), use a 0.9 or 1.0 load factor, whichever generates the critical load case.

Extreme Event III

Supplement <Article 3.4.1> with the following:

Extreme Event III – Load combination relating to a structural member or component failure as it relates to the System Redundancy Evaluation for Steel Twin Tub Girders as discussed in Section 5.6.3.5.

Supplement <Table 3.4.1-1> with the following:

Supplement of <Table 3.4.1-1 – Load Combinations and Load Factors>

Load Combination Limit State	DC DW EH EV ES EL 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:

Supplement of <Table 3.4.1-2 – Load Factors Permanent Loads, γ_p>

Type of Load, Foundation Type	Load Factor	
	Maximum	Minimum
DC: Components and Attachments for the evaluation of system redundancy as specified in Section 5.6.3.5, for Extreme Event III only	1.10	0.90

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

3.8.2 Vehicular Collision Loads

Replace <Article 3.6.5> with the following:

Abutments and retaining walls:

- Due to the soil behind abutments and retaining walls, design of the collision force is not required.

Bents:

- Investigate bents for collision when located within a distance of 30.0 feet to the edge of roadway. A bridge deck adjacent to the column is an adjacent roadway.
- Investigate the need for vehicular collision design for the final condition after all construction is completed, not during construction phases with temporary traffic conditions.
- Investigate the need for vehicular collision design by determining the annual frequency for a bridge bent or pier to be hit by a heavy vehicle (AF_{HPB}) or optionally the annual frequency of bridge collapse (AF_{BC}).
- Do not design bents and piers for collision when AF_{HPB} is less than 0.001. Use the following equations to determine AF_{HPB}:

  The annual frequency for a bridge bent or pier to be hit by a heavy vehicle: $AF_{HBP} = 2(ADTT)(P_{HBP})365$

  $ADTT$ = the number of trucks per day in one direction

  The annual probability for a bridge pier to be hit by a heavy vehicle:

$$P_{HBP} = \begin{cases} 3.457 \times 10^{-9}, & \textit{for undivided roadways in tangent and} \\ & \textit{horizontally curved sections} \\ 1.090 \times 10^{-9}, & \textit{for divided roadways in tangent sections} \\ 2.184 \times 10^{-9}, & \textit{for divided roadways in horizontally} \\ & \textit{curved sections} \end{cases}$$

- If AF_{HBP} is greater than 0.001 the designer may optionally calculate AF_{BC} using <Equation C3.6.5.1-1>. If AF_{BC} is less than 0.001, the bents do not need to be designed for vehicular collision.

- When designing 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, founded, 54 inch tall, MASH Test Level 5 approved concrete rail if the top edge of the traffic face of the rail is within 3.25 feet from the columns. The back face of the rail should be offset from the column to allow dynamic displacement of the rail without the rail impacting the column.
    - Protect with a structurally independent, founded, 42 inch tall, MASH Test Level 5 approved concrete rail if the top edge of the traffic face of the rail is between 3.25 feet and 10 feet from component.
    - Protect with a 42-inch-tall single slope concrete barrier (or 42-inch-tall, MASH Test Level 5 approved barrier equivalent) if more than 10 feet from component.
- When the design choice is to provide structural resistance, the design must meet the following requirements:
    - Design the bent for an equivalent static force of 600 kips. The force is acting in a direction of zero to 15° with the edge of the pavement in a horizontal plane. Apply the force at a distance from 2.0 feet to 5.0 feet above ground, whichever produces the critical effect being analyzed. Analyze the column and the connections to the foundation and bent cap.
    - The load may be applied as a point load or may be distributed over an area deemed suitable for the size of the structure and the anticipated impacting vehicle, but not greater than 5.0 feet wide by 2.0 feet high centered around the assumed impact point.
    - If the bent cannot be designed for the 600-kip load, then redirection will be required.
    - See Section 4.14 Vehicular Collision for design information.

3.8.3 Railroad Collision

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

3.8.4 Vessel Collision Loads

TxDOT requires that all bridges crossing waterways with documented commercial vessel traffic comply with <Article 3.14>. For widening of the existing structures, at a minimum 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.

3.8.5 Earthquake Effects

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

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 in Brewster, Presidio, Jeff Davis, Culberson, Hudspeth or 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 in Brewster, Presidio, Jeff Davis, Culberson, Hudspeth or El Paso counties. In these locations, evaluate the structure for earthquake effects as required by <Article 3.10>. Contact the Bridge Division (BRG_Design@txdot.gov) for guidance.

3.9 Temperature Gradient

The following superstructure types do not require analysis for temperature gradient as shown in <Article 3.12.3>:

- Simply supported prestressed beams.
- Cast-in-place slab and girder spans.
- Cast-in-place slab spans.
- Spliced I-shaped girders.
- Steel I-beams.
- Steel plate-girders.
- Steel tub-girders.

Chapter 4 General Requirements

4.1 Overview

This chapter documents policy on general requirements for bridge design. For criteria on other transportation structures, see Chapter 7 for concrete culverts and Chapter 8 for ancillary structures.

4.2 Bridge Geometry

See Chapter 5 for beam orientation information and limits on beam spacing.

Match bridge geometry on the Bridge Layout to the Plan and Profile sheets.

Place cross slope transitions off the bridge where possible. If cross slope transitions must be accommodated on the bridge deck, place transitions at a bent. Use linear cross slope transitions to facilitate screeding.

Avoid having multiple cross slope breaks. At a gore where multiple alignments merge or separate, multiple cross slope breaks may be used.

4.3 Bridge Width

Refer to the *TxDOT Bridge Project Development Manual* for bridge width considerations.

For wide bridges, design bearing pads for transverse thermal movements in addition to the typical longitudinal movements.

Place outermost lateral restraints no more than 80 feet apart, to prevent excess restraint under thermal movements.

For abutment caps and bent caps when the length of cap exceeds 80 feet, a construction joint may be used. Locate the joint close to a dead load inflection point. Place the joint such that a clear distance to the bearing seat is 3 inches minimum.

Wide bridge decks may have issues with concrete shrinkage and deck cracking. A longitudinal open joint may be used when the bridge width exceeds 100 feet. Treat each section of the superstructure as an independent bridge for lateral constraint.

Phasing of the bridge deck allows most of the concrete shrinkage to occur in the separate pours for the narrower phase widths. Instead of a longitudinal open joint, design the bearing pads for the transverse and longitudinal thermal movements.

On divided highways, use separate bridges for each direction of traffic. A single bridge for both directions of traffic may be used if the median is less than 30 feet wide between inside lane edges.

When using a concrete median barrier with a longitudinal open joint, place the barrier on one side of the open joint.

Alternately to median barrier, use bridge rails placed back-to-back on either side of the open joint. Provide details for transitioning from back-to-back bridge rails to the concrete median barrier, if applicable.

Commentary

For more information on median barriers, see Roadway Standards, Single Slope Concrete Barrier on Bridge Deck (SSCB(1)) and Concrete Safety Barrier (F-Shape) on Bridge Deck (CSB(3)).

In the past, median barriers were allowed to be over the joint. This caused the barrier toe to crack. This is no longer acceptable for new construction.

4.4 Vertical Clearance

Design bridges to meet or exceed the vertical clearances required in the *TxDOT Roadway Design Manual*.

Maintain sufficient vertical clearance for inspection access.

If project circumstances, such as construction phasing, require temporary vertical clearance to be less than the required vertical clearance:

- Verify that a reduced temporary vertical clearance is acceptable.
- Consider using beam types that can better withstand an over-height impact prior to deck placement, e.g. slab beams, box beams, and X-beams.
- If using beam types that are less able to withstand an over-height impact prior to deck placement, e.g. TxGirders, steel beams, and steel girders, adjust the construction phasing to minimize the duration of time that girders are in place prior to placement of the concrete deck.

4.5 Lateral Restraint of Bridge Superstructures on Substructure

4.5.1 General

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

4.5.2 Bridges Crossing Water Features

Provide details for LR shear keys on abutment and bent caps of I-girder 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 low chord 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.

4.5.3 I-Girder Bridges

4.5.3.1 Crossing Water Features:

Location of a LR shear key is at the discretion of the designer; however, LR shear keys are typically located 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.

Commentary

Refer to the Bridge Standard Shear Key Details for I-Girders (IGSK) for detailing information.

4.5.4 Spread Box Beam and U Beam Bridges

4.5.4.1 Grade Separations:

Include LR shear keys on bent and abutment caps of spread box beams and U beams superstructures when the roadway has single direction cross slope. Location of a LR shear key is at the discretion of the Engineer; however, LR shear keys are typically located between the exterior and first interior beam on the high side of cross slope.

Commentary

Refer to the Bridge Standard Shear Key Details for X-Beams (XBSK) for detailing information. Design and details shear keys for U-Beams.

LR shear keys are required on bent and abutment caps of spread box beams and U beams because these beams are placed in a rotated position to match the roadway cross slope. The center of gravity of the beam is not located over the center of the bearing pad and causes the beam to slide downhill. When the roadway cross slope is crowned, beams on opposite sides of the crown provide opposing forces and limit the lateral movement of the superstructure. When the roadway cross slope is single direction, the superstructure is not self-restrained and LR shear keys are needed to limit lateral movement.

Consider using shear keys when the bridge may experience significant vibration from trains or pile driving.

4.5.4.2 Crossing Water Features:

Location of a LR shear key is at the discretion of the designer; however, shear keys are typically located between the exterior and first interior beam on the upstream side of bridge.

Commentary

Refer to the TxDOT Bridge Standard Shear Key Details for X-Beams (XBSK) for design and detailing information. Design and detail shear keys for U-Beams.

4.5.5 Adjacent Slab Beam and Box Beam Bridges

Additional lateral restraint measures are not required for slab beam and box beam superstructures unless the bent cap does not have an earwall.

Commentary

In general, the standard earwalls on the abutments and bents are adequate for preventing transverse movement of the beams. Under extreme loading conditions (high water velocities and frequent overtopping), larger earwalls may be warranted. Design earwalls for the design flood loading and for the buoyancy of the beam.

4.5.6 Steel Beam or Girder Bridges

4.5.6.1 Crossing Water Features:

Provide lateral restraints on steel bridges crossing water features. For additional considerations and guidance, refer to the *TxDOT Preferred Practices for Steel Bridge Design, Fabrication, and Erection.*

4.6 Phased Construction

4.6.1 General

Do not use span, bent, or abutment standard detail sheets for phased structures.

Design each phase of construction, including the final configuration, according to this manual.

4.6.2 Superstructure

When designing the beams, account for all temporary loading such as temporary rails as permanent loads for that phase. Design beams so that they meet all requirements for all phases of construction.

The beam located under the phase line will have less dead load deflection than the other beams constructed at the same time. This beam will not deflect additionally when the remainder of the slab is cast, due to the added stiffness of the cured slab. When calculating haunch for the beam along the phase line, use the dead load deflection from the initial slab weight. Do not use the full dead load deflection due to the full slab weight (initial and final).

Bearing seat elevations may be lowered in later phases to account for the potential for higher than predicted cambers. There is no way to adjust the roadway grade in subsequent phases to accommodate high camber girders.

Phased superstructures may vary beam spacing for each phase and for the beams near the phase line.

Load rating of the existing structure is required if the phasing scheme removes portions of the existing structure. Acceptable load rating limits for phased construction of existing structures should be discussed with the District where the work is performed.

Account for traffic needs and the placement of any temporary barriers when selecting phase line locations. If the clear distance between the back of the barrier and the edge of the slab is less than 2 feet, pin the barrier to the deck. If possible, allow a 2-foot buffer when placing the temporary barrier on new bridge deck to avoid the need for installing pins in the new bridge deck.

When constructing next to an existing structure (such as for phased replacements), provide enough space between the existing structure and the new construction to accommodate the following:

- splicing of the deck reinforcement.
- the portion of the beam that extends beyond the edge of slab.
- the portion of bent or abutment that extends past the beam edge.
- any reinforcing of the bent or abutment that extends into the next phase.
- formwork.

For TxGirders, place the phase line as shown in Figure 4.6-1. Other restrictions are as follows:

- Do not place a phase line in the middle or at the edge of a precast panel as shown in Figure 4.6-2.
- Do not place the phase line closer than 10 inches from the beam edge, to allow for the use of precast panels in the future phase.
- Place the phase line a minimum of 4 inches past the centerline of the girder, so that the horizontal interface reinforcement is cast into the initial construction phase of the slab.
- An alternative option is to place the phase line between two beams. Treat the slab between the beam and the phase line as an overhang. Do not allow the use of panels in this bay.

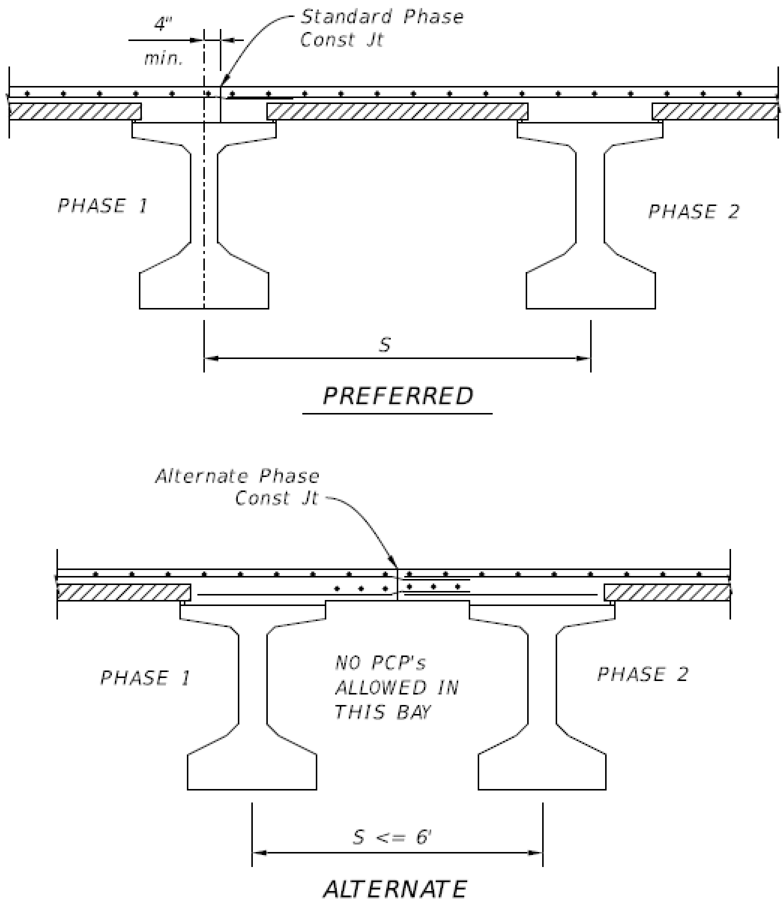


Figure 4.6.2-1: Phasing for TxGirders

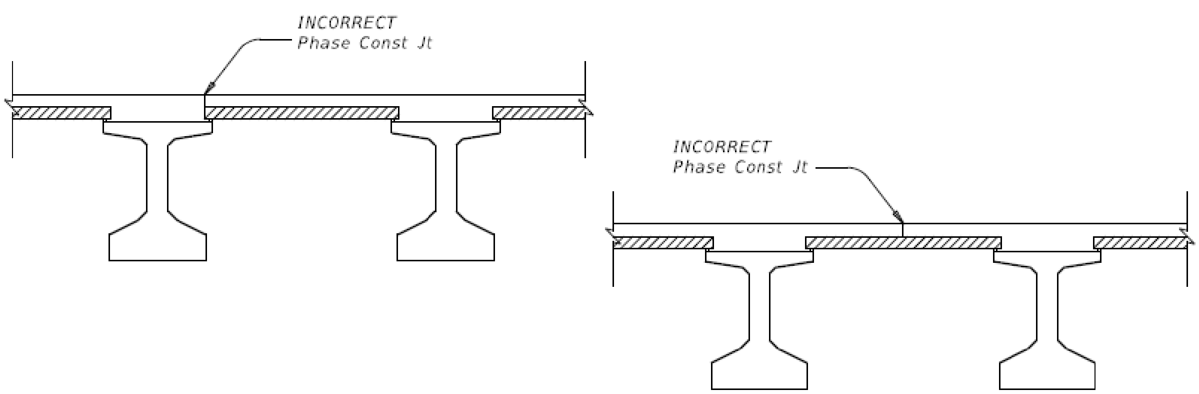


Figure 4.6.2-2: Incorrect Phasing for TxGirders

For adjacent slab or box beam superstructures, place the phase line at the edge of the beam, as shown in Figure 4.6-3 and Figure 4.6-4. Do not place a phase line within the top flange of a Slab Beam or adjacent Box Beam as shown in Figure 4.6-5.

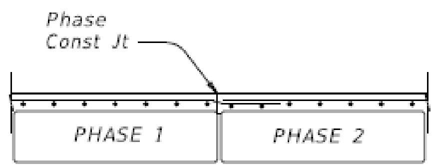


Figure 4.6.2-3: Phasing for Slab Beams

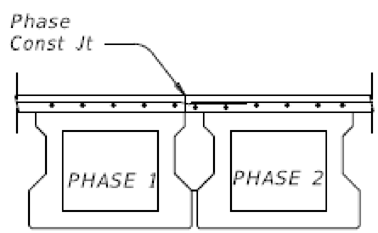


Figure 4.6.2-4: Phasing for Adjacent Box Beams

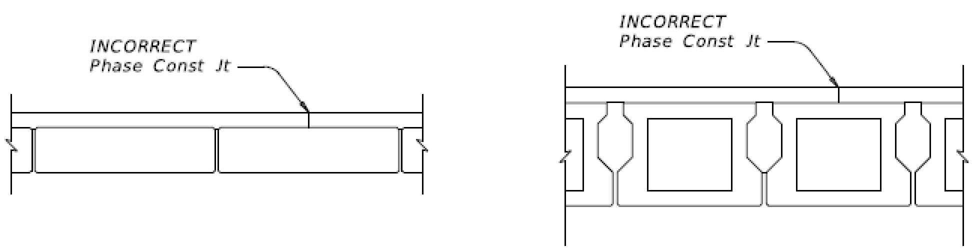


Figure 4.6.2-5: Incorrect Phasing for Slab and Box Beam

For spread box beams (X-Beams), place the phase line as shown in Figure 4.6-6. Other restrictions are as follows:

- Place the phase line along the top flange of the beam. If the phase line is located along the top flange of the beam, much of the beam will be under the initial phase of construction.
- Do not place the phase line closer than 10 inches from the beam edge for X Beams, to allow for the use of precast panels in the future phase.
- Alternately place the phase line between two beams. Treat the slab between the beam and the phase line as an overhang. Do not allow the use of panels in this bay.

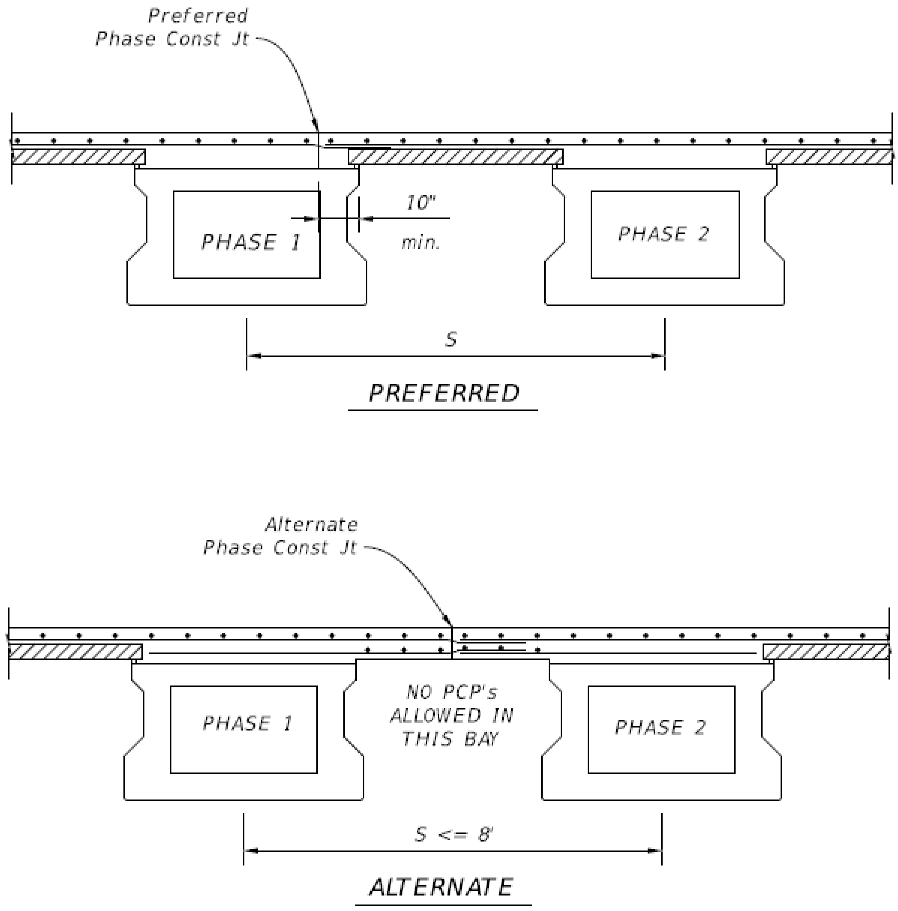


Figure 4.6.2-6: Phasing for X-Beams

4.6.3 Substructure

When designing bents and abutments to be continuous after phasing, design for all stages of construction (including temporary loads) and the final configuration. Design for locked in dead load effects between phases. Select flexural and shear reinforcement so that loading in all phases can be supported.

Phased construction may require additional beam dowels, and wide structures may require beam dowels to be moved to inside girders. See the guidelines for abutments, interior bents, and inverted-tees for maximum recommended space between beam dowels. If a full depth joint is used between the phases, each component may require a beam dowel.

Additional lateral restraint (LR shear keys) may be needed for phased structures.

Design bents and abutments that have full depth joints at the phase line as individual components.

In most cases, the phase line in an abutment or interior bent will be offset from the phase line for the slab. The phase line should not be under a beam or within a bearing seat. Extend the abutment or interior bent past the slab phase line to provide support for the beam or girder. The phase line should be a minimum of 3 inches from the bearing seat or edge of beam, whichever is further.

When phasing an abutment or an interior bent, provide enough space between the existing structure and the new construction to accommodate splicing of the reinforcement and formwork. Account for how the next phase of construction will be impacted by the placement of phase lines and reinforcement that extends beyond the phase line. Drilled shaft or pile for the next phase may lie within the length necessary for splicing. Avoid having splices that overlap drilled shaft or pile locations to facilitate construction.

If unable to provide enough room to splice the reinforcement through traditional lapping, use welded splices or mechanical couplers. In some cases, a combination of couplers/welded splices and traditional lapping may be utilized for elements with varying bar sizes. Extend reinforcement that will be spliced by welds or mechanical couplers beyond the end of the cap by at least 1-foot.

As an alternative to splicing or welding the reinforcement, a full depth joint may be used at the phase line. For abutments, if a full depth joint is used, limit the space between abutments to 1-inch. Use bituminous fiber to fill the gap between the phases. Use a PVC waterstop across the space along the full height of the back of the cap and the backwall.

For bent caps, the full depth open joint at the phase line should be at least 1-foot wide to allow for forming of the adjacent phases. Individual bent caps would support each phase.

When selecting column or drilled shaft/pile spacing, set the distance from face of column or drilled shaft/pile to the phase line between 0.5 and 4 feet. Overhangs greater than 4 feet can result in high negative moments and permanent deflection of the overhang under loading. The construction of additional phases will not remove this deflection.

Phased construction of abutments or bents may require that columns or drilled shafts be spaced at irregular intervals.

Offset old bent lines and new bents by at least 5 feet, if possible, to keep from fouling foundations on the existing structure. Existing structures may have battered piling and may be widened with battered piling. Account for potential battered pile conflicts with wingwalls.

4.7 Widenings

4.7.1 General

As appropriate, apply new bridge design requirements to various elements for bridge widenings.

Perform a load rating analysis and complete a condition survey before plans are started. Load ratings must conform to the current edition of the *TxDOT Bridge Inspection Manual*.

Refer to the *TxDOT Bridge Project Development Manual* for additional requirements for minimum load ratings and considerations related to modifications of existing structures.

Review the Bridge Inspection follow-up action recommendations and the condition surveys and include all necessary recommendations unless otherwise directed by TxDOT.

Design widened portions for HL-93 loading in accordance with the *AASHTO LRFD Bridge Design Specifications* and this manual.

Show load rating of the existing structure to remain as well as design loads for the widening on the bridge plans, for example,

Superstructure Inv/Opr Ratings= X.XX/Y.YY (widening)

Superstructure Inv/Opr Ratings= HSXX / HSXX (existing)

Evaluate a range of geometric aspects when developing bridge widenings including:

- Vertical clearance in the existing and widened condition.
- Relative horizontal clearances to lower roadways or reconfigured roadways.
- Maintaining bridge skew, if possible.

- In stream crossings, the relative stream geometry with respect to the widened bridge.

Plan and elevation information based on existing plans may not reflect in-situ conditions. Provide surveyed information where possible and ensure plans require contractor verification by adding "Field verify existing dimensions and elevations prior to ordering materials and beginning work" to the plan sheets where appropriate.

Consult existing plans (as-builts, rehabilitation, etc.) for such items as bearing conditions, expansion joint conditions, continuous beams, and lateral restraint that might influence design and details for the widened condition.

Match the expansion joint system of the existing structure unless it has been retrofitted with a header joint. Evaluate condition of existing joints and replace full width if damage exists. Strip seals in sealed expansion joints should be replaced over the full combined width.

Widen existing approach slabs with approach slabs that meet the current standard thickness and design.

4.7.2 Superstructure Widenings

For superstructures that have a cast-in-place composite bridge deck in the existing and widened construction, provide bridge details that:

- Use the current standard deck thickness in the new construction regardless of the thickness of the existing bridge deck.
- Examine the load carrying capacity of existing exterior beam in the proposed condition when determining location of proposed beams.
- Locate slab breakback line at the centerline or towards the outside of the centerline of the existing beam to ensure composite action is retained in the broken back condition.

- Account for panel placement and strand extension for prestressed concrete panels used in the first beam bay adjacent to the existing. If insufficient room exists, restrict the use of panels. Timber or permanent metal deck forms may be used as forming in this bay.
- Do not widen a traditionally designed deck with an empirically designed deck.
- If applicable, use a thickened end slab in the widened construction regardless of original diaphragm details.

4.7.2.1 Cast-in-Place Slab Span Widenings

Widen cast-in-place slab spans with cast-in-place slab spans.

Skewed slabs with the main reinforcing perpendicular to the bents may be weakened if the edge beam is removed under traffic. Shore the edge under this condition. Alternatively, keep the curb in place and grout dowels into the existing slab edge and then place the widening with reinforcing parallel to the centerline of the roadway. Remove the curbs after the new slab has cured.

Do not widen Flat Slab (FS) Bridges. FS Bridges should be replaced. FS Bridges have a structural curb acting as an edge beam that cannot be removed without shoring the span.

4.7.2.2 Cast-in-Place Concrete Girder Widenings

Widen cast-in-place concrete girder spans with pretensioned concrete I-girders, pretensioned concrete box beams, or pretensioned concrete slab beams. If vertical clearance is an issue, use box beams or slab beams.

4.7.2.3 Pan Form Girder Widenings

Widen pan form girders with pretensioned concrete box beams or pretensioned concrete slab beams. If widening with pan form girders, contact the Bridge Division (BRG_Design@txdot.gov) to discuss the circumstance and possible alternatives.

4.7.2.4 Simple Span Steel I-Beam Widenings

Widen simple span steel I-beams with steel I-beams or pretensioned concrete I-girders. If depth, framing, or aesthetics are issues, use steel I-beams.

4.7.2.5 Continuous Steel I-Beam Widenings

Widen continuous steel I-beam units with steel I-beams or pretensioned concrete beams. Simple span pretensioned concrete I-girders with the slab continuous are commonly used.

4.7.2.6 Cantilever/Drop-In Steel I-Beam Widenings

Widen cantilever/drop-in steel I-beam units with continuous steel I beams or simple span pretensioned concrete I-girders. For simple span pretensioned concrete I-girders, use a longitudinal open joint connected to the existing and proposed joints. Other solutions that eliminate the longitudinal open joint are recommended.

4.7.2.7 Continuous Steel Plate Girders Widenings

Widen continuous steel plate girder units with continuous steel plate girders or with pretensioned concrete I-girder if the span is 150 feet or less.

4.7.2.8 Pretensioned Concrete I-Beams and I-Girders Widenings

Widen pretensioned concrete I-beam and pretensioned concrete I girder spans and units with pretensioned concrete I-girders.

4.7.2.9 Prestressed Concrete Box Beams Widenings

Widen pretensioned concrete box beam spans and units with pretensioned concrete box beam.

Evaluate the condition of these bridges carefully to determine if widening is feasible. Consult with Bridge Division for guidance on preferred details.

Commentary

Existing box beams prior to the mid-1980's had specific details that showed the cast-in-place end diaphragms partially integral with the substructure.

Existing box beam bridges may not have a composite concrete slab and only have an asphalt overlay.

4.7.2.10 Prestressed Concrete Slab Beams Widenings

Widen prestressed concrete slab beam spans and units with prestressed concrete slab beams.

Slab beams have a limited slab thickness and potential damage in removal operations may occur.

4.7.3 Substructure Widenings

Foundations of widened bridge sections can be of a different type than the existing foundations. Design foundations to the same depth or deeper than the existing foundation.

Account for minimum clearances for pile driving leads or drilled shaft equipment. Provide a minimum clearance between the new foundation and the existing foundation of two times the least dimension of the existing/proposed foundation. If a foundation element is outside of the bounds of supporting the substructure due to the minimum clearance requirements, a new foundation may be installed closer, provided the following is evaluated:

- Group effect interaction of the existing foundations and the new foundations.

- Load carrying capacity of existing foundation while it remains in service during installation of adjacent foundation.

If the existing substructure that is to be widened has a tie beam or web wall, incorporate a tie beam or web wall into the substructure widening, unless the tie beam or web wall will cause measurable negative impacts to scour, debris accumulation, or excess stream force.

4.7.3.1 Abutment Widenings

If there is an existing backwall and the bearing area is sufficient for the new superstructure, match the face of the existing backwall and connect by breaking back and providing a minimum splice length of horizontal steel.

4.7.3.2 Interior Bent Widenings

Widen interior bents using one of the following:

- Independent non-connected bents with two or more supporting columns.
- Independent non-connected bents with one supporting column.
- A connected bent with one or more columns using a bent cap shear connection.
- A connected bent with one or more columns using a bent cap shear and moment connection. A moment and shear connections will require breaking back part of the existing bent. Verify that the localized removal of concrete in the existing bent cap does not compromise the structural resistance of the existing bent cap relative to existing structural loads (dead and live loads).

4.8 Approach Slabs

Use of approach slabs is encouraged.

Supporting an approach slab on wingwalls is strongly discouraged. An approach slab supported by the wingwalls and abutment backwall requires a non-standard design for the approach slab and supporting elements.

Use of cement stabilized abutment backfill (CSAB) wedges in the zone behind the abutment is encouraged.

Commentary

Approach slabs provide a smooth transition between the bridge deck and the roadway pavement.

Approach slabs minimize the effect of differential settlements between the bridge abutment founded on shafts or piles and the embankment fill.

Approach slabs also decrease the live load effect on the abutment backwall, which decreases the lateral load and overturning of the abutment cap.

The standard approach slab is designed to be supported on the backfill and accommodates an unsupported length of 10 feet from the backwall to the fill to address the possibility of uncompacted backfill or wash out behind the abutment.

Approach slabs, when properly placed, provide a seal to prevent water from seeping into the soil behind the abutment backwall and in front of the wingwall. Sealing the gap between the approach slab and the wingwalls is very important.

Compaction of backfill is difficult and loss of backfill material can occur. CSAB can solve the problem of difficult compaction behind the abutment, and it resists the moisture gain and loss of material common under approach slabs.

4.9 Corrosion Protection

Determine the requirement for corrosion protection by consulting with the District and the *TxDOT's Structure Design – Corrosion Protection Guide*.

Consult the Bridge Division about recommended corrosion protection measures in marine environments.

Commentary

TxDOT's Structure Design – Corrosion Protection Guide provides statewide and district specific recommendations and the district for project-related protection measures.

The TxDOT Statewide planning map provides corrosion risk mapping at a county level.

4.10 Expansion Joints

4.10.1 Material

See Item 454, "*Bridge Expansion Joints*" for material requirements.

Do not use aluminum joint components.

4.10.2 Geometric Constraints

Provide bridge deck continuity by providing multi-span units to minimize the number of joints.

Use sealed or drained expansion joints in de-icing zones.

If there is a concern with snowplows or deicing, an unarmored concrete header with seal may be used instead of an armor joint (AJ) or sealed expansion joint (SEJ).

Open joints may be used for stream crossing structures in salt-free zones without environmental concerns.

Use sealed joints for all grade separation structures.

The following are available joints for new construction:

Pourable Seal (TYPE A):

- Preferred joint for low traffic volume off-system structure.
- Used for spans or units no longer than 100 feet.

Open Armor Joint (ARMOR JOINT):

- Not allowed on CIP slab spans, in de-icing zones, grade separations, or over steel beams.
- Minimum opening = 0-inch, Maximum opening = 2 inches.
- Total movement = 2 inches.

Pourable Seal (Sealed Armor Joint (ARMOR JOINT)(SEALED)):

- Minimum opening = 0.75 inches, Maximum opening = 2.25 inches.
- Total movement = 1.5 inches.

Mechanically Bonded Sealed Expansion Joint (SEJ-M and SEJ-S(O)):

- Preferred joint for freight corridors or other high truck volume roadways.
- SEJ-M - No overlay, SEJ-S(O) – With ±2-inch overlay.
- Minimum slab and overhang thickness = 6.5 inches.
- Available in two sizes:

  4-inch

  - Minimum opening = 0-inch, Maximum opening = 4 inches.
  - Total movement = 4 inches.

  5-inch

  - Minimum opening = 0-inch, Maximum opening = 5 inches.
  - Total movement = 5 inches.

Bonded Strip Seal Sealed Expansion Joint (SEJ-B):

- No overlay.
- Minimum slab and overhang thickness = 6.5 inches.
- Minimum opening = 0-inch, Maximum opening = 4 inches.
- Total movement = 4 inches.

Header Type Joints with Seal:

- Only use when there is a thickened slab end.
- If there is no thickened slab end, use ACP overlay or other means to mitigate the encroachment of the header material below the bridge slab.
- Minimum joint opening = 0.5 inches, Maximum joint opening = 3 inches.

- The joint seal can accommodate +/- 50% movement. The joint opening shown on the plans at 70°F typically should not be less than 1 inch or greater than 2 inches. The minimum joint opening is to keep the sealant from squeezing upward and being abraded. The maximum joint opening is to limit the actual width to reduce the potential for impact loading at the span end, which could cause the header to fail.

Finger Joint and Modular Bridge Joint Systems (MBJS):

- For expansion movements greater than 5 inches, finger joints or modular joints may be used. These systems tend to be high maintenance. Consult with the Bridge Division on the use of these joint types.

4.10.3 Design Criteria

Base the total movement required through a bridge deck expansion joint on a temperature range of 10°F to 110°F for concrete bridges and 0°F to 120°F for steel bridges. Alternatively, use the temperature contour maps in <Article 3.12.2>.

Use coefficients of expansion equal to 6.0×10^{-6} for concrete and 6.5×10^{-6} for steel.

When placed on a skew, sealed expansion joints (SEJ) have a reduced ability to accommodate longitudinal movement. Calculate reduced movement range by multiplying the joint size by cosine (skew).

When integral or semi-integral bents are used, design for the effect column stiffness has on the distribution of thermal movement.

Adjacent box beam and slab beams have 5-inch slab thickness. These beams have a required block out in the beam that allows an increased slab thickness for the AJ and SEJ joints.

4.10.4 Detailing

Expansion joints are predesigned and shown on standard drawings except for finger joints and modular bridge joint systems (MBJS), which require project specific details.

Additional or supplemental joint details may be required for widenings or bridges constructed in phases.

For projects with inverted-tee bent caps, use two joints (one at each face of the inverted-tee stem) unless otherwise directed by the District.

4.11 Non-Contact Lap Splices

4.11.1 General

Use a non-contact lap splice when columns are stepped or drilled shafts frame into dissimilarly shaped or sized columns.

4.11.2 Detailing

Use a reinforcement confinement factor (λ_{rc}) of 1.0 when calculating the required lap length (l_s).

Provide a total splice length: $l_{ns} = l_s + s_{sp}$, where s_{sp} equals the maximum lateral spacing between longitudinal reinforcement transferring the force.

Provide transverse reinforcement along the provided splice length that satisfies the following requirements:

- For laps confined by spiral reinforcing, provide transverse reinforcement that satisfies <Equation 5.10.8.4.2a-1>.
- For laps confined by stirrups, provide transverse reinforcement spaced no wider than the maximum allowed spacing of transverse reinforcement (s_{max}) for each face of the column. Use the following equation to determine the maximum allowed spacing of transverse reinforcement for each face:

$$s_{max} = \frac{n_{tr} A_{tr} f_{ytr} l_s}{A_{Tl} f_{ul}}$$

where:

n_{tr} = number of legs of column transverse reinforcement, perpendicular to face of the column.

A_{tr} = area of column transverse reinforcement (in^2).

f_{ytr} = specified minimum yield strength of column transverse reinforcement (ksi).

l_s = standard required splice length (in).

A_{Tl} = Total area of longitudinal reinforcement in tension at face of the column (in^2).

f_{ul} = ultimate strength of longitudinal reinforcement (ksi).

- Do not violate minimum spacing requirements for concrete consolidation as outlined in <Article 5.10.3.1>.

4.12 Steel-Reinforced Elastomeric Bearings

4.12.1 Materials

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

Use a shear modulus range of 95 to 175 pounds per square inch (psi) for design, using the least favorable value for the design check.

Use 0.105-inch-thick steel shims.

Do not use adhesives between bearings and other components.

4.12.2 Geometric Constraints

See standard drawings for standard pad details. The bearings pads shown on the standards were not designed for

- skews exceeding those shown on the associated standard span details (45° skew for TxGirders and 30° skew for steel beams, slab beams, box beams, and X-beams).
- multi-span units with the slab continuous over interior bents that consist of more than 3 spans.
- multi-span units with the slab continuous over interior bents that have short end spans.
- beam spacings less than 6 feet or greater than 10 feet.

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

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

Disregard the minimum edge cover for the steel reinforcement in <Article 14.7.6.1>. Provide ⅛-inch to ¼-inch minimum edge cover.

Commentary

Rectangular pads are preferred over round pads, which makes it harder to satisfy rotation requirements. Round pads are typically needed for highly skewed bent caps.

4.12.3 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 dead load condition (the lightest predicted dead load) when checking against slip.

Use Load Combination Service I for all gravity loads.

Use load factor for uniform temperature (TU) of 1.0.

Ignore limit on S_i^2/n in <Article 14.7.6.1>.

When using multi-span units, the overall length of the unit, end span length, skew, and roadway grade influence the bearing pad design. The end span length can control the design of the bearing pad and limit the overall unit length.

Commentary

Expanding length of prestressed concrete beam units can be taken as ½ total unit length. For highly skewed bridges and very wide bridges, take expanding length on a diagonal between slab corners to obtain the most unfavorable expansion length.

4.12.4 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

  $$\frac{\theta(0.8L)}{2}$$

  where:

  L = length of the bearing pad, perpendicular to the bridge longitudinal axis.

  θ = the total rotation (radians).

  Estimate compressive deflection using <Figure C14.7.6.3.3-1>.

- Calculate total rotation for dead (DL) and live load (LL) plus 0.005 radians for construction uncertainties as required by <Article 14.4.2.1>. Take maximum live load rotation as:

  $$\frac{4\Delta}{SpanLength}$$

  where Δ is midspan LL deflection.

  Check bearing pad slip as follows:

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

where:

Gr = beam grade (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 area of elastomer (in^2).

$\Delta_{s(allow)}$ = 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>.
- For Design Method A in <Article 14.7.6>, the preferred shape factor (S) is between 10 and 12.

Commentary

The preference is to use standard bearing pad designs/details where possible. Prior to detailing custom pads, the Engineer should verify if a standard pad will work.

4.12.5 Detailing

Use standard drawings for guidance on detailing custom bearing pad designs.

4.13 Strut-and-Tie Method

4.13.1 Structural Analysis

Do not use strut-and-tie for girders and bent caps that fall within the standard practice reflected in the Bridge Standards.

Use strut-and-tie modeling or other refined analysis methods when designing footings, dapped beam ends, post-tensioning anchorage zones, deviation diaphragms, bents that use high load bearings, and other special designs.

Do not use strut-and-tie for lightweight concrete design.

Commentary

Place nodes at applied loads and reactions. More nodes can be added as long as the tension ties are located where reinforcement is normally placed. In general, nodes need to be located at the center of the tension ties and compression struts. If there is sufficient concrete in the connected member the strut can be considered within both members, such as in the case where a column connects to a footing. If the nodes are placed where the two members meet, use the lower concrete strength of the two members for strut checks.

A 3-dimensional truss can be broken into multiple 2-dimensional trusses to design tension ties. When analyzing the 2-dimensional trusses, use the same reactions as the 3-dimensional truss, but recalculate the applied loads so equilibrium is satisfied. Use a 3-dimentional truss to verify the strut-to-tie angles satisfy the geometric limitations in <Article 5.8.2.2> and to check the strut-to-node interface stresses.

4.13.2 Design Criteria

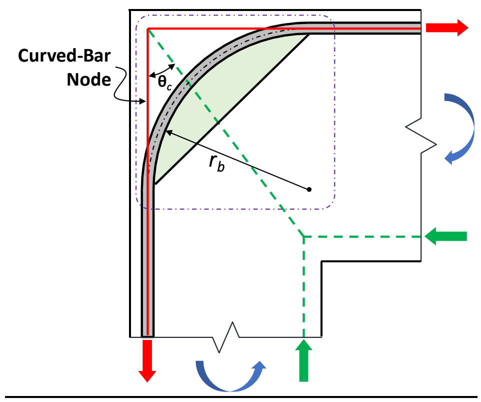
For members designed with strut-and-tie, check the calculated shear, using the Service I Load Combination, against the resistance from <Equation C5.8.2.2-1>.

For members designed with strut-and-tie, check two-way shear at concentrated loads and reactions per <Article 5.12.8.6.3>.

Use strut bearing lengths proportional to the amount of load carried by the strut at a node <Figure C5.8.2.2-4>.

Supplement <Article 5.8.2.2> with the following:

- Curved-bar: nodes formed by the bend region of a continuous reinforcing bar (or bars) where two ties extending from the bend region are intersected by a strut or the resultant of two or more struts. The bar bend should be specified to be 90°.



(d) Curved-bar Node

r_b = bend radius at the inside of the bar (or bars) of a curved-bar node. (in)

θ_c = the smaller of the two angles between the axis of the strut (or the resultant of two or more struts) and the ties extending from a curved-bar node.

Supplement to <Figure 5.8.2.2-1 – Nodal Geometries>

Design and detail curved-bar nodes in accordance with the following requirements:

- Design bend radius of the bar bend to meet the following equation but shall not be less than half the minimum diameter specified in <Article 5.10.2.3>.

$$r_b \geq \frac{A_{ts} f_y}{b_s \nu f'_c} \psi_c$$

where:

A_{ts} = the area of non-prestressed bars in the tie (in^2).

f_y = the yield strength of the bars (ksi).

b_s = the width of the diagonal strut (or resultant struts) framing into the curved-bar node (in).

ν = the efficiency factor for curved-bar node, taken as 0.5.

f'_c = the compressive strength of concrete (ksi).

ψ_c = cover factor, taken as the greater of $2d_b/c_c$ and 1.0.

where:

d_b is the bar diameter (in).

c_c is clear cover to the side face (in).

- If multiple layers of longitudinal reinforcing bars are used, make the bend radius of the bar bend of every layer the same.
- If bundled bars are used, enlarge the bend radius by a factor of 1.25.
- Reinforce struts framing into the curved-bar node with appropriate orthogonal crack control reinforcement in accordance with <Article 5.8.2.6>.

Commentary

The bottom tension tie reinforcement does not have to be located within a 45° distribution angle from the supporting edge (i.e. no more than cover dimension (d_c) away from the member on either side). Research (Yi et al., 2021) indicates reinforcing bars both outside and inside the 45° bandwidth can be included as the total tie area if the reinforcement is sufficiently anchored.

Conservatively assume the width of a strut in a CCC node (h_s) as the height of the compression block as a starting point. Adjust h_s as needed as long as the truss model geometry is modified to place the strut at the midpoint of h_s.

References:

Yi, Y., H. Kim, R. A. Boehm, Z. D. Webb, J. Choi, H.C. Wang, J. Murcia-Delso, T.D. Hrynyk, and O. Bayrak, "3D Strut-and-Tie Modeling for Design of Drilled Shaft Footings," Report No. FHWA/ TX-21/0-6953-1, Center for Transportation Research, The University of Texas at Austin, Austin, TX., (https://library.ctr.utexas.edu/ctr-publications/0-6953-1.pdf), 2022.

Wang, H.-C., A. Daou, A. Alomari, J. Choi, T. Dinh, Y. Shao, Y. Yi, Z. Webb, and O. Bayrak, "The Development of Knowledge in the Application of Strut-and-Tie Method", Report No. FHWA/TX-23/0-7039-1, Center of Transportation Research, The University of Texas at Austin, Austin, TX., (https://library.ctr.utexas.edu/ctr-publications/0-7039-1.pdf), 2023.

4.14 Vehicular Collision

Evaluate bents for collision when located within a distance of 30.0 feet to the edge of roadway. When designing for collision, either redirect the collision load or provide structural resistance.

When the design choice is to redirect the collision load, follow the requirements given in Section 3.8.2.

When the design choice is to provide structural resistance, design for the 600-kip equivalent static load as described in Section 3.8.2. Design the column and the connection to the bent cap and the connection to the foundation to withstand the collision force in shear and flexure. Neglect the collision load when determining the loads for foundation design.

Use of a vehicular deflection wall between the columns is permitted if necessary. Design vehicular deflection wall assembly (columns plus wall) for the 600-kip equivalent static load as described in Section 3.8.2.

No further analysis is required for columns and drilled shafts with a gross cross-sectional area of no less than 40 square feet; a minimum thickness of 5 feet and column transverse reinforcement is composed of at least No. 4 ties at 12-inch maximum spacing or a No. 4 spiral at 9-inch maximum pitch.

Commentary

Refer to the TxDOT Bent Protection Guide for more information on vehicle collision loads, determining probability of impact or collapse, and the design of vehicle deflection walls.

Chapter 5 Superstructure Design

5.1 Overview

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

5.2 Concrete Deck Slabs

This section documents policy on concrete deck slabs on spread beams and on adjacent beams.

5.2.1 Concrete Deck Slabs on Spread Beams

5.2.1.1 Materials

Use normal weight Class S concrete (f_c' = 4.0 ksi, where f_c' is the concrete compressive strength).

Use lightweight concrete Class S Concrete (f_c' = 4.0 ksi) when needed to reduce substructure loading and with prior approval of the Bridge Division.

Use Grade 60 reinforcing steel or deformed welded wire reinforcement (WWR) meeting the requirements of American Society for Testing and Materials (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 high performance concrete (HPC) concrete and/or use one of the following types of corrosion resistant reinforcement (refer also to Item 440, "Reinforcement for Concrete"):

- Epoxy-coated reinforcing steel meeting the requirements of ASTM A775 or A934.
- Epoxy-coated deformed welded wire reinforcement (WWR) meeting the requirements of ASTM A884 Class A or B.
- 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*. Specify a minimum modulus of elasticity for GFRP of 7,500 ksi in the plans.
- Low-carbon and low-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.

5.2.1.2 Geometric Constraints

Use 8.5-inch-deep minimum concrete deck slabs. Use of thinner concrete deck slabs is not permitted.

Provide 2.5 inches clear cover to the top mat reinforcing bars and 1.25 inches clear to the bottom mat reinforcing bars. Provide 2 inches cover to bar ends.

Maximum overhang is 3.33 feet 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. Maximum overhang dimensions are based on preventing excessive torsion on fascia girders and on rail crash testing.

Minimum overhang is 0.5 feet from top beam or flange edge except for wide flange pretensioned concrete I-girder and pretensioned concrete spread box beams, which have a 0-foot minimum overhang. The minimum overhang dimension is based on the ability to use overhang brackets.

Commentary

Deck slabs less than 8.0 inches thick cannot be used with TxDOT's standard prestressed concrete panels because they do not provide enough practical room above a 4-inch panel. An 8.5-inch-thick deck provides added durability and allows grinding if the roadway grade needs adjustment.

5.2.1.3 Design Criteria

Where applicable, use the empirical design of <Article 9.7.2> with the following requirements:

- For pretensioned concrete I girders and steel I beam/girders maximum beam spacing is 10 feet. For spread box beams and steel tub girders maximum spacing is 10 feet from centerline of exterior web.
- Top mat reinforcement is No. 4 bars at 9-inch maximum spacing (0.27 square inches/foot) in both transverse and longitudinal directions. Place transverse bars closest to the top deck slab surface. In the overhangs, place No. 5 bars extending 2 feet minimum past fascia girder web centerline between each transverse No. 4 bar.
- Bottom mat reinforcement is No. 4 bars at 9-inch maximum spacing (0.27 square inches/foot) in both transverse and longitudinal directions. Place transverse bars closest to the bottom deck slab surface.
- For continuous beams (i.e. steel plate girders and concrete spliced girders), where the longitudinal tensile stress in the concrete deck due to either the factored construction loads or Load Combination Service I in <Table 3.4.1-1> exceeds $0.9f_r$ (modulus of rupture):
    - Provide longitudinal reinforcement with a total cross-sectional area of at least one percent of the total cross-sectional area of the cast-in-place portion of the concrete deck.
    - Design and detail for the worst-case between the full depth cast in place deck or patrial depth cast in place deck over panels when both options are allowed on the span sheet.
    - Extend longitudinal reinforcing steel at least one development length (l_d) past the point of contraflexure.
- 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 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 the Prestressed Concrete Panels (PCP) standard drawing.

- The overhang need not extend past the exterior girder more than 6 inches beyond the flange edge (0 inches for wide flange pretensioned concrete I girder, pretensioned concrete spread slab beams, and pretensioned concrete 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. Use the traditional deck design of <Article 9.7.3> for steel twin tub girder spans designed for redundancy per, Section 5.6.3. Use the traditional design of <Article 9.7.3> with the following requirements:

- The minimum amount of longitudinal reinforcement in the top mat is No. 4 bars at 9-inch maximum spacing for these deck designs.
- For continuous beams (i.e. steel plate girders and concrete spliced girders), where the longitudinal tensile stress in the concrete deck due to either the factored construction loads or Load Combination Service I in <Table 3.4.1-1> exceeds $0.9f_r$:

    Provide longitudinal reinforcement with a total cross-sectional area of at least one percent of the total cross-sectional area of the cast-in-place portion of the concrete deck.

    Design and detail for the worst-case between the full depth cast in place deck or patrial depth cast in place deck over panels when both options are allowed on the span sheet.

    Extend longitudinal reinforcing steel at least one development length (l_d) past the point of contraflexure.

To account for reduced wheel load distribution at transverse slab edges adjacent to expansion joints, strengthen the slab by increasing its depth and reinforcement 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. The Thickened Slab End Details standard drawings are:

- IGTS for TxGirders.
- SGTS for Steel I-beams and Plate Girders.
- XBTS for Spread Box Beams.

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

Commentary

TxDOT's slabs, on beams and girders, are based on empirical deck design, also known as isotropic deck design. This decision was based on various research projects; collaboration with construction and maintenance experts; and past performance.

For continuous beams (i.e. steel plate girders and concrete spliced girders), there is a requirement to provide at least one percent longitudinal reinforcement to control cracking in negative moment regions. This one percent is based on the cast-in-place portion of the deck, for example, for 4.5-inch cast-in-place deck over a 4-inch panel would be designed for one percent of the 4.5-inch depth and for a full depth, 8.5-inch deck would be designed for one percent of the 8.5-inch depth.

For an 8.5-inch concrete deck, the one percent requirement can be achieved for both the full depth cast-in-place deck and the partial depth cast-in-place over panels by providing additional No. 5 longitudinal bars spaced at 9-inch maximum located between the No. 4 longitudinal bars at 9-inch maximum spacing.

5.2.1.4 Detailing

With simple-span construction, minimize expansion joints by creating multi-span units with the slab continuous over interior bents. At bents without expansion joints, locate a control joint or construction joint in the deck. Refer to Section 4.12 for bearing pad constraints.

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

Breakback deck slab corners at a dimension of 2 feet when skew is more than 15°. Place the corner break point at least 1 inch and preferably 3 inches from the toe of any concrete parapet into which an expansion joint is upturned.

The use of panels is encouraged. The use of PCPs is prohibited for certain applications:

- Curved steel girder bridges. Use a monolithic deck on these units because of the complicated interaction between the deck, the curved girders, and the diaphragms.
- Bridge widenings. PCPs are not typically allowed in the bay adjacent to the existing structure because it is usually not possible to set the panels properly on the existing structure. PCPs can be used in the other bays when the widening involves multiple girders.
- Steel girders with narrow flanges. Girders with flanges less than 12 inches wide make PCP use difficult because the shear studs conflict with the panels. Standard details allow shear studs to be skewed across the flange width to facilitate the use of PCPs where sufficient flange width is available.

Commentary

TxDOT has standard detail sheets for prestressed concrete panels (PCP) on prestressed concrete beams and structural steel beams. PCP designs are highly standardized and follow the AASHTO LRFD Bridge Design Specifications.

The panels support the slab dead load. The composite PCP/cast-in-place slab cross section resists slab live load moments. Distribution reinforcing is not required.

Strands are located at mid-depth in a 4-inch-thick panel. When 5-inch-thick panels are needed consider locating strands at 2-inches above the bottom. This makes panel construction simpler for the fabricators.

PCPs are reinforced with ⅜-inch or ½-inch diameter Grade 270 strands at 6 inches on center. Regardless of size, the strand is stressed to 14.4 kips and an assumed total loss of 25 kips per square inch (ksi) is used for design. These limits were set to reduce the cracking of the panels. The panels are generally not wide enough to develop larger strands, so the design is based on the amount that can be developed rather than full development.

For short panels, usually 5 feet or less, No. 4 Grade 60 reinforcing steel can be substituted for the strands. Reinforcing steel instead of prestressing strands is required in panels shorter than 3.5 feet to prevent splitting.

The panel width (span) can be adjusted in each panel to accommodate flaring beams, but this requires diligence on the contractor’s part to ensure that the panels are set in the correct location.

5.2.2 Concrete Deck Slabs on Adjacent Beams

5.2.2.1 Materials

Use normal weight Class S concrete (f_c' = 4.0 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 for regions where bridge decks are exposed to de-icing agents and/or saltwater spray with regularity. If thus required, use high performance concrete (HPC) concrete and/or use one of the following types of corrosion resistant reinforcement (refer also to Item 440, "Reinforcement for Concrete"):

- Epoxy-coated reinforcing steel meeting the requirements of ASTM A775 or A934.
- Epoxy-coated deformed welded wire reinforcement (WWR) meeting the requirements of ASTM A884 Class A or B.
- 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*. Specify a minimum modulus of elasticity for GFRP of 7,500 ksi in the plans.
- Low-carbon and low-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.

5.2.2.2 Geometric Constraints

Use 5-inch-deep minimum concrete deck slabs placed on a minimum 5.5-inch-deep top flange. Use of thinner concrete deck slabs or top flanges are not permitted.

Use 2.5-inch top clear cover.

5.2.2.3 Design Criteria

For transverse reinforcement, use No. 5 bars spaced at 6 inches maximum.

For longitudinal reinforcement, use No. 4 bars spaced at 12 inches maximum.

5.2.2.4 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.

5.3 Prestressed Concrete Beams

This section documents policy on the design of prestressed concrete beams. Section 5.3.1 applies to all prestressed concrete beam types. Section 5.3.2 through Section 5.3.5 provide specific beam type requirements.

5.3.1 Prestressed Concrete Design Requirements

This section provides requirements that apply to all prestressed concrete beams and girders.

5.3.1.1 Materials

Use normal weight Class H concrete with a

- Minimum f'_{ci} = 4.0 ksi and f'_c = 5.0 ksi.
- Maximum f'_{ci} = 6.0 ksi and f'_c = 8.5 ksi.
- where f'_{ci} is the initial concrete compressive strength and f'_c the final.

Any exceptions to these limits must be approved in writing by the TxDOT Bridge Division.

For adjacent box beams, use Class S concrete (f'_c = 4.0 ksi) for shear keys between the beams.

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

Commentary

Previously, TxDOT policy held designs to a firm maximum of 6.0 ksi compressive strength at time of release of prestressing tension. The *TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges* now include provisions for limiting cement content to prevent issues related to alkali silica reaction (ASR), delayed ettringite formation (DEF), autogenous shrinkage, and other problems related to high cement content.

Designers may make a request to TxDOT Bridge Division for an exception for cases in which a slightly higher release strength will allow for longer spans to keep bents out of channels, a girder line can be eliminated, or some other discernible benefit to TxDOT.

TxDOT will consider these requests on a case-by-case basis. While each case will be considered, designers should generally aim for 6.0 ksi maximum release strength to minimize negative impact to the precast fabrication production rates.

5.3.1.2 Geometric Constraints

The minimum number of beams or 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 beams or girders per span may be used.

5.3.1.3 Structural Analysis

Distribute the weight of one railing to no more than three girders, applied to the composite cross section (except for U-Beams).

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>.

Use section properties given on the beam or girder standard drawings. For the composite section, use gross section properties.

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.

5.3.1.4 Design Criteria

Use a relative humidity of 60% regardless of the location of the bridge.

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

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

Initial compression stress up to $0.65f'_{ci}$ (ksi) is allowed. $0.7f'_{ci}$ (ksi) is not 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 accounted for in the analysis.

Final tension stress up to $0.19\,\lambda\sqrt{f'_c}$ (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.
- $0.40f'_c$ 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 $0.24\lambda\sqrt{f'_c}$ is allowed for checking concrete stresses during deck and diaphragm placement.

Use an effective strand stress after release of $0.75f_{pu}$ - Δf_{pES}, where Δf_{pES} is the prestress loss due to elastic shortening.

Use the General Procedure as provided by <Article 5.7.3.4.2> to determine shear resistance. Do not use provisions of <Appendix B5>.

Calculate required stirrup spacing for No. 4 Grade 60 bars according to the <Article 5.7>. Change stirrup spacing as shown on standard drawing for prestressed beams only if analysis indicates the inadequacy of the standard design.

Only apply the requirement in <Article 5.7.3.5> from inside face of support to inside face of support. Do not calculate from the inside face of support to the end of the beam.

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 (c) and friction (μ) factors as provided in <Article 5.7.4.4> as follows:

- c = 0.28 ksi
- μ = 1.0
- K1 = 0.3
- K2 = 1.8 ksi

Replace <Equation 5.4.2.3.2-2> with the following:

$$k_s = 1.45 - 0.13\left(\frac{V}{S}\right) > 0.0$$

Compute deflections due to slab weight and composite dead loads assuming the girder and slab to have the same modulus of elasticity (E_c). 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.

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:

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

Shrinkage loss:

$$\Delta f_{pSR} = E_p \left(\frac{140 - H}{4.8 + f_{ci}'}\right)(4.4 \times 10^{-5})$$

Creep loss:

$$\Delta f_{pCR} = 0.1 \left(\frac{195 - H}{4.8 + f_{ci}'}\right)\left(\frac{E_p}{E_{ci}}\right)\left(f_{cgp} + 0.6 \Delta f_{cd}\right), where\ \Delta f_{cd} = -\left(\frac{M_{sd} e_p}{I_g}\right)$$

Relaxation loss:

$$\Delta f_{pR} = \frac{2 f_{pt}}{K_L}\left(\frac{f_{pt}}{f_{py}} - 0.55\right)$$

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.

Commentary

The reason for setting the humidity at 60% is that this is about an average relative humidity for Texas and is consistent with designs shown on standard drawings. In addition, the beams could be cast at a location with a different humidity than the bridge location.

5.3.1.5 Detailing

Grouping beam designs is encouraged.

For each design, show optional design parameters for maximum top flange stress, bottom flange stress, and ultimate moment due to all design loads on the plans. The fabricator has the option to use other strand arrangements if design parameters are satisfied by the prestress and concrete strength selected.

For grade separation structures, use the same girder depth for the full length of structure for economies of scale and aesthetic reasons. Stream crossing structures may have different types and sizes of girders for purposes of economy.

Optimize girder spacing in each span. Maintaining constant girder spacing for the full length of structure is not necessary.

Commentary

For bridges with multiple spans, it is more economical to group beam designs. This allows the fabricator to limit the number of different types of beams to fabricate. Beams can also be grouped across various bridges in the same project. For grouping beams, TxDOT recommends grouping beams where there is a difference of 4 strands or less. Provide a unique beam design where there is a difference of 6 strands or more.

5.3.1.6 Software

It is strongly recommended to use BridgeLink (PGSuper) for beam design. Refer to PGSuper Design Guide for further guidance about using PGSuper for beam design.

PGSuper calculates live load distribution factors. BridgeLink (PGSuper) can be downloaded from the TxDOT and FHWA Engineering Software website.

5.3.2 Pretensioned Concrete I-Girders

This section provides the additional requirements that apply to all prestressed concrete I-girders (TxGirders and Wide Flange I-Girders).

5.3.2.1 Geometric Constraints

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

I-girders are oriented plumb to the deck. Bearing seat elevations are defined by one point located at the intersection of the CL of beam and CL of bearing. See Appendix A for information on the framing of TxGirders, calculation of haunch, and the calculation of bearing pad tapers.

5.3.2.2 Structural Analysis

Use section properties given on the Prestressed Concrete I-Girders Details (IGD) and Prestressed Concrete Wide Flange I-Girder (WF-IGD) standard drawings.

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.

When prestressed concrete deck panels or stay-in-place metal forms are allowed, design the beam using the basic slab thickness (8.5 inches).

5.3.2.3 Design Criteria

Add and drape strands in the order shown on the Prestressed Concrete I-Girder Non-Standard Designs (IGND) and Prestressed Concrete Wide Flange I-Girder Designs (WF-IGND) standard drawing.

Draping strands is the preferred method to reduce tensile stresses at the end of the beam. Use hold-down points shown on the standard drawing IGD and WF-IGD.

Straight strand designs with and without debonding are permitted as an alternate to draping provided the stress and other limits noted below are satisfied.

Do not use both draped strands and straight strands with debonding.

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 feet.
- Not more than 50% 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 (3 feet, 6 feet, 9 feet, 12 feet, or 15 feet).

TxDOT standard I-girders reinforced as shown on the IGD and WF-IGD standard drawings are adequate for the requirements of <Article 5.9.4.4>.

For TX girders, restrict the maximum number of strands in girders as follows:

- TX28 thru TX40, 44 – 0.6” strands.
- TX46 thru TX70, 54 – 0.6” strands.

If the strand limit needs to be exceeded, contact the Bridge Division (BRG_Design@txdot.gov) to discuss alternatives. If more strands are decided on, then the girder’s ends need to be designed for splitting resistance <Article 5.9.4.4.1>.

Commentary

There are physical limits on the total prestress force a fabricator’s production lines can handle and too many strands can overwhelm the mild reinforcement meant to control bursting and spalling cracks in the girder end regions. The software might indicate the design works, but the design can be impractical or impossible to construct.

5.3.2.4 Detailing

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

Flare as few beams as necessary when framing a flared span to avoid fabricating several geometrically different precast deck panels.

5.3.3 Pretensioned Concrete Adjacent Slab Beams

This section provides the additional requirements that apply to all prestressed concrete slab beams placed side-by-side (adjacent slab beams).

5.3.3.1 Geometric Constraints

Limit skew to 30°. The standard beam details do not accommodate skews larger than 30°. Larger skews may result in beam twist and uneven bearing on the pads.

When two or more different widths are needed, use the largest width as the exterior beams.

Avoid slab overhangs on adjacent slab beams. Choose beams and gap sizes so that the edge of the slab corresponds to the edge of the top flange of the exterior beams.

The minimum gap between adjacent slab beams is 0.5 inches and the maximum gap is 3.31 inches. A preferable gap range is 1 inch to 1.5 inches.

Place adjacent slab beams parallel to the roadway surface when the cross slope is constant or rotated to the average cross slope of a span in a transition area. Slope the top of the cap to match the rotation of the beams.

When both a vertical curve and skew exist, a complex planar relationship develops between the skewed bottom of the slab beam, bearing pad, and bent or abutment cap. A stepped cap elevation arrangement may be required.

Prestressed concrete adjacent slab beams are rotated to minimize torsion from the slab load. Groups of beams are rotated together, with an elevation point at each side of the group of beams. The cap is sloped from one elevation point to the next, but flat transverse to the cap centerline.

Use a three-pad system with adjacent slab beams. The forward station end of the beam sits on a single elastomeric bearing pad while the back station end sits on two smaller pads.

Bearing pads are tapered along the centerline of the beam, and flat transverse to the beam centerline. Bearing pad tapers are calculated similar to X-Beams, see Appendix B for example bearing pad taper calculations.

Do not use dowels for lateral restraint. Provide lateral restraint by ear walls located at the ends of each abutment and interior bent cap. Provide a ½-inch gap between the ear wall and the outside edge of the exterior beam.

Do not use bearing seats with adjacent slab beams. Set the pads directly on top of the cap. Provide top of cap elevation points at the following locations:

- Coinciding with the outer edge of the exterior beams.
- At changes in cap slope.
- At changes in cap elevations, such as left and rights elevations at steps.

Commentary

Adjacent slab beams are not appropriate for use on curved structures and should be avoided on flared structures. The complexity of the geometry required to frame the bridge increases dramatically as the degree of curvature exceeds 1° or 2°.

Framing is complicated in cross-slope transition areas and skewed bridges. Orient the beams to minimize the variation in slab thickness both longitudinally and transversely along the span. This may require stepping the cap at some joints so that adjacent beams not only have a different slope but also sit at a different elevation. Elevation points may be required as often as every joint in some situations. The forward half of an interior bent cap may have a different elevation than the back half at some locations.

Where the elevation needed for the beam group at the left of the elevation does not equal the elevation needed for the beam group at the right of the elevation point, a vertical step in the cap is needed. Where this elevation difference is small, the average elevation is used. For simplicity we call these elevation points steps on the plans.

For skewed bridges, the requirement to bevel the bearing pads to match the beam slope on the Elastomeric Bearing Details sheet will not result in the top of the pad surface and the bottom of beam surface being parallel. The actual calculations and fabrication of pads for each particular skewed case is complex. Given the small area of the pads, experience with slab beams and the nearly parallel surfaces, the pads should be able to deform sufficiently to accommodate the mismatches.

5.3.3.2 Structural Analysis

Use section properties given on the Prestressed Slab Beams standard drawings.

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>.

5.3.3.3 Design Criteria

Add and debond strands in the order shown on the Slab Beam Non-Standard Design (PSBND) standard drawing.

Debond strands in 3-foot increments at beam ends if necessary to control stresses at release. If the strand size is larger than 0.6” diameter, base section increments on <Article 5.9.4.3.3>.

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

- Debond no more than 50% of the total number of strands.
- Debond no more than 50% of the number of strands in that row.
- Replace Restriction B with, not more than 50% 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 (3 feet, 6 feet, 9 feet, 12 feet, and 15 feet).
- Do not design for Restriction E.
- Replace Restriction G with 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 feet.

TxDOT standard slab beams satisfy <Article 5.7.4> and <Article 5.9.4.4>.

See Section 5.3.1 for other design criteria.

5.3.4 Pretensioned Concrete Adjacent Box Beams

This section provides the additional requirements that apply to all prestressed concrete box beams placed side-by-side (adjacent box beams).

5.3.4.1 Geometric Constraints

Limit skew to 30°. The standard beam details do not accommodate skews larger than 30°. Larger skews may result in beam twist and uneven bearing on the pads.

When two or more different widths are needed, use the largest width as the exterior beams.

Avoid slab overhangs on box beams. Choose beams and gap sizes so that the edge of the slab corresponds to the edge of the top flange of the exterior beams.

The minimum gap between adjacent box beams is 1 inch, and the maximum gap is 2 inches.

Place adjacent box beams parallel to the roadway surface when the cross slope is constant or rotated to the average cross slope of a span in a transition area. Slope the top of the cap to match the rotation of the beams.

When both a vertical curve and skew exist, a complex planar relationship develops between the skewed bottom of the box beam, bearing pad, and bent or abutment cap. A stepped cap elevation arrangement may be required.

Prestressed concrete adjacent box beams are rotated to minimize torsion from the slab load. Groups of beams are rotated together, with an elevation point at each side of the group of beams. The cap is sloped from one elevation point to the next, but flat transverse to the cap centerline.

Use a three-pad system with adjacent box beams. The forward station end of the beam sits on a single elastomeric bearing pad while the back station end sits on two smaller pads.

Bearing pads are tapered along the centerline of the beam, and flat transverse to the beam centerline. Bearing pad tapers are calculated similar to X-Beams, see Appendix B for example bearing pad taper calculations.

Do not use dowels for lateral restraint. Provide lateral restraint by ear walls located at the ends of each abutment and interior bent cap. Provide a ½-inch gap between the ear wall and the outside edge of the exterior beam.

Do not use bearing seats with box beams. Set the pads directly on top of the cap. Provide top of cap elevation points at the following locations:

- Coinciding with the outer edge of the exterior beams.
- At changes in cap slope.
- At changes in cap elevations, such as left and rights elevations at steps.

Commentary

Box beams are fabricated using a two-stage monolithic casting. The bottom slab is cast in the first stage, and the sides and top are cast in the second stage while the slab concrete is still plastic. In addition, cardboard void forms are no longer permitted. All interior voids must be formed with polystyrene. Void drain holes are installed at the corners of the bottom slab during fabrication.

Adjacent box beams are not appropriate for use on curved structures and should be avoided on flared structures. The complexity of the geometry required to frame the bridge increases dramatically as the degree of curvature exceeds 1° or 2°.

Framing is complicated in cross-slope transition areas and skewed bridges. Orient the beams to minimize the variation in slab thickness both longitudinally and transversely along the span. This may require stepping the cap at some joints so that adjacent beams not only have a different slope but also sit at a different elevation. Elevation points may be required as often as every joint in some situations. The forward half of an interior bent cap may have a different elevation than the back half at some locations.

Where the elevation needed for the beam group at the left of the elevation does not equal the elevation needed for the beam group at the right of the elevation point, a vertical step in the cap is needed. Where this elevation difference is small, the average elevation is used. For simplicity we call these elevation points steps on the plans.

For skewed bridges, the requirement to bevel the bearing pads to match the beam slope on the Elastomeric Bearing Details sheet will not result in the top of the pad surface and the bottom of beam surface being parallel. The actual calculations and fabrication of pads for each particular skewed case is complex. Given the small area of the pads, experience with box beams and the nearly parallel surfaces, the pads should be able to deform sufficiently to accommodate the mismatches.

5.3.4.2 Structural Analysis

Use section properties given on the Prestressed Box Beams standard drawings.

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.

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>.

5.3.4.3 Design Criteria

Add and debond strands in the order shown on the Prestressed Concrete Box Beam Non-Standard Design (BBND) standard drawings.

Debond strands in 3-foot increments at beam ends if necessary to control stresses at release. If the strand size is larger than 0.6” diameter, base section increments on <Article 5.9.4.3.3>.

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

- Debond no more than 50% of the total number of strands.
- Debond no more than 50% of the number of strands in that row.
- Replace Restriction B with not more than 50% 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 (3 feet, 6 feet, 9 feet, 12 feet, and 15 feet).
- Up to 75 percent of debonded strands may be used for the total number, the number of strands per row, and number terminated in a section as long as principal stress at or near the transfer length is designed for per <Article 5.9.2.3.3>, regardless of the concrete strength.
- Do not design for Restriction E.
- Replace Restriction G with 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 feet.

- For multi-web sections having bottom flanges, replace Restriction J with:

  Uniformly distribute debonded strands.

  Bond the outer-most strand in each row.

TxDOT standard box beams satisfy <Article 5.7.4> and <Article 5.9.4.4>.

Use shear keys between the beams for all adjacent box beam bridges. Do not account for composite action between beams and shear keys in computing live load distribution factors, nor for strength, stress, or deflection calculations.

See Section 5.3.1 for other design criteria.

5.3.5 Pretensioned Concrete Spread Box Beams

This section provides the additional requirements that apply to all spread prestressed concrete box beams (X-Beams).

5.3.5.1 Geometric Constraints

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

Use a three-pad system with spread box beams. Typically, the forward station end of the beam sits on a single elastomeric bearing pad while the back station end sits on two smaller pads.

Rotate box beams to accommodate the average cross slope of a given span. The depth of slab haunch at the left and right top edges of the beam may differ.

Pretensioned concrete spread box beams are defined by left and right edge of the bearing seat pedestal and are balanced individually to eliminate torsion from the slab load.

Left and right bearing seat elevations are located at the intersection of the edges of bearing seats with centerline bearings.

The bearing seat is sloped from one side to the next, but flat transverse to the cap centerline. Taper bearing pads along the centerline of the beam, and flat transverse to the beam centerline.

The main location for rotating X-beams is at the centerline of the bottom of the beam. This method allows the X-beams to be framed as vertical members. This results in the beam spacings dimensioned on the span details and beam layouts matching the beam spacings shown on the substructure details. A construction note is recommended on the span details stating, "Beam spacing shown is measured at bottom of beam. Beam spacing at top of beam may vary due to cross slope of box beams."

Use the same minimum haunch value for all box beams in a given span if reasonable to do so.

See Appendix B for information on the framing of X-Beams and the calculation of haunch and bearing pad tapers.

Commentary

Box beams are fabricated using a two-stage monolithic casting. The bottom slab is cast in the first stage, and the sides and top are cast in the second stage while the slab concrete is still plastic. In addition, cardboard void forms are no longer permitted. All interior voids must be formed with polystyrene. Void drain holes are installed at the corners of the bottom slab during fabrication.

An alternate for rotating X-beams is at the centerline of the top of the beam. This prevents spacing at the top of the beam from varying due to the cross slope of the beam and, thus, simplifies slab formwork dimensions for construction.

5.3.5.2 Structural Analysis

Use section properties given on the Prestressed Concrete X-Beams standard drawings.

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 feet, 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 feet, 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.

When prestressed concrete deck panels or stay-in-place metal forms are allowed, design the beam using the basic slab thickness (8.5 inches).

5.3.5.3 Design Criteria

Add and debond strands in the order shown on the Prestressed Concrete X-Beam Non-Standard Designs (XBND) standard drawings.

Debond strands in 3-foot increments at beam ends if necessary to control stresses at release. If the strand size is larger than 0.6 inches in diameter, base section increments on <Article 5.9.4.3.3>.

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

- Debond no more than 50% of the total number of strands.
- Debond no more than 50% of the number of strands in that row.
- Replace Restriction B with not more than 50% 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 (3 feet, 6 feet, 9 feet, 12 feet, and 15 feet).

- Up to 75 percent of debonded strands may be used for the total number, the number of strands per row, and number terminated in a section as long as principal stress at or near the transfer length is designed for per <Article 5.9.2.3.3>, regardless of the concrete strength.
- Do not design for Restriction E.
- Replace Restriction G with 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 feet.
- For multi-web sections having bottom flanges, replace Restriction J with:
  - Uniformly distribute debonded strands.
  - Bond the outer-most strand in each row.

TxDOT standard X-beams satisfy <Article 5.7.4> and <Article 5.9.4.4>.

See Section 5.3.1 for other design criteria.

5.3.5.4 Detailing

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

Use thickened slab ends at all expansion joints with non-inverted-tee bents. See the Thickened Slab End Details standard drawing.

If inverted-tee caps are used and are sloped to match the sloping face of the box beam, use a 4:1 slope normal to the centerline of the bent.

Use slab dowels to provide lateral restraint when constructing box beams with inverted-tee bents. These dowels are located at the top of the inverted-tee stem and are in a slotted pipe to allow for expansion and contraction of the unit. Typically, only one dowel is placed at the centerline of every beam. Slab dowels need to be placed on only one side of the centerline of the bent.

5.3.6 Pretensioned Concrete U-Beams

The use of U-beams requires Bridge Division approval.

U-beam drawings may be requested from Bridge Division (BRG_Design@txdot.gov). These drawings must be signed and sealed.

This section provides the additional requirements that apply to all prestressed concrete U-beams.

5.3.6.1 Geometric Constraints

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

Use a three-pad system with U-beams. Typically, the forward station end of the beam sits on a single elastomeric bearing pad while the back station end sits on two smaller pads.

Rotate U-beams to accommodate the average cross slope of a given span. The depth of slab haunch at the left and right top edges of the beam may differ.

Pretensioned concrete U-beams are defined by left and right edge of the bearing seat pedestal and are balanced individually to eliminate torsion from the slab load.

Left and right bearing seat elevations are located at the intersection of the edges of bearing seats with centerline bearings.

The bearing seat is sloped from one side to the next, but flat transverse to the cap centerline. Taper bearing pads along the centerline of the beam, and flat transverse to the beam centerline.

The main location for rotating U-beams is at the centerline of the bottom of the beam. This method allows the U-beams to be framed as vertical members. This results in the beam spacings dimensioned on the span details and beam layouts matching the beam spacings shown on the substructure details. A construction note is recommended on the span details stating, "Beam spacing shown is measured at bottom of beam. Beam spacing at top of beam may vary due to cross slope of U-beams."

Use the same minimum haunch value for all U-beams in a given span if reasonable to do so.

U-beams are framed the same as X-beams. See Appendix B for information on the framing of X-Beams and the calculation of bearing pad tapers.

Commentary

An alternate for rotating U-beams is at the centerline of the top of the beam. This prevents spacing at the top of the beam from varying due to the cross slope of the beam and, thus, simplifies slab formwork dimensions for construction.

5.3.6.2 Structural Analysis

For U-beams, 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 inches. They are located as close as 10 feet 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 Prestressed Concrete U-Beam drawings.

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 feet, 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 feet, 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.

When prestressed concrete deck panels or stay-in-place metal forms are allowed, design the beam using the basic slab thickness (8.5 inches).

5.3.6.3 Design Criteria

Add and debond strands in the order shown on the Prestressed Concrete U-Beam Non-Standard Designs (UBND) drawings.

Debond strands in 3-foot increments at beam ends if necessary to control stresses at release. If the strand size is larger than 0.6 inches in diameter, base section increments on <Article 5.9.4.3.3>.

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

- Debond no more than 50% of the total number of strands.
- Debond no more than 50% of the number of strands in that row.
- Replace Restriction B with not more than 50% 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 (3 feet, 6 feet, 9 feet, 12 feet, and 15 feet).

- Up to 75 percent of debonded strands may be used for the total number, the number of strands per row, and number terminated in a section as long as principal stress at or near the transfer length is designed for per <Article 5.9.2.3.3>, regardless of the concrete strength.
- Do not design for Restriction E.
- Replace Restriction G with 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 feet.
- For multi-web sections having bottom flanges, replace Restriction J with:
  - Uniformly distribute debonded strands.
  - Bond the outer-most strand in each row.

See Section 5.3.1 for other design criteria.

5.3.6.4 Detailing

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

Use thickened slab ends at all expansion joints with non-inverted-tee bents. See the Thickened Slab End Details drawing.

If inverted-tee caps are used and are sloped to match the sloping face of the U-beam, use a 4:1 slope normal to the centerline of the bent.

Use slab dowels to provide lateral restraint when constructing U-beams with inverted-tee bents. These dowels are located at the top of the inverted-tee stem and are in a slotted pipe to allow for expansion and contraction of the unit. Typically, only one dowel is placed at the centerline of every beam. Slab dowels need to be placed on only one side of the centerline of the bent.

5.4 Post-tensioned Superstructures

This section documents policy on the design of post-tensioned superstructures, including spliced girders and segmental spans.

5.4.1 Spliced Precast Girders

5.4.1.1 Materials

Use normal weight Class H or Class H (HPC) concrete for girder elements:

Precast Elements:

- minimum f'_{ci} = 4.0 ksi and f'_c = 6.0 ksi.
- maximum f'_{ci} = 6.0 ksi and f'_c = 10.0 ksi.

Cast in Place Elements:

- maximum f'_c = 6.0 ksi.

Refer to Section 5.2.1 for requirements of concrete deck slab.

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*", 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 permanent tendons that are stressed at the precast yard must be grouted prior to transport.

5.4.1.2 Geometric Constraints

The minimum number 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 feet, a minimum of 4 girders is required.
- U-Section: 2 girders.

For I-Sections:

- I-girders are oriented plumb to the deck. Bearing seat elevations are defined by one point located at the intersection of the CL of beam and CL of bearing.

For U-Sections:

- Rotate U-sections to accommodate the average cross slope of a given span. The depth of slab haunch at the left and right top edges of the beam may differ.
- U-sections are defined by left and right edge of the bearing seat pedestal and are balanced individually to eliminate torsion from the slab load.
- Left and right bearing seat elevations are located at the intersection of the edges of bearing seats with centerline bearings.
- The bearing seat is sloped from one side to the next, but flat transverse to the cap centerline. Taper bearing pads along the centerline of the beam, and flat transverse to the beam centerline.
- The main location for rotating U-sections is at the centerline of the bottom of the beam. This method allows the U-sections to be framed as vertical members. This results in the beam spacings dimensioned on the span details and beam layouts matching the beam spacings shown on the substructure details. A construction note is recommended on the span details stating, "Beam spacing shown is measured at bottom of beam.

Beam spacing at top of beam may vary due to cross slope of spliced U-beams."

Commentary

An alternate for rotating U-sections is at the centerline of the top of the beam. This prevents spacing at the top of the beam from varying due to the cross slope of the beam and, thus, simplifies slab formwork dimensions for construction.

5.4.1.3 Structural Analysis

Girder designs should meet the following requirements:

- Base the self-weight of the girder on a minimum of 160 pounds per cubic foot (pcf).
- For I-Sections: Distribute the weight of one railing to no more than three girders, applied to the composite cross section.
- For U-sections: 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.
- Haunch concrete placed on top of the girder may be included 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.
- Live load distribution can be determined from one of the following methods:
    - Must conform to <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 <Article 4.6.2.2.2> and <Article 4.6.2.2.3> when used in conjunction with a line girder analysis; or

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.

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 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>.

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.

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

Analysis must account for 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.

Commentary

Bridge Division suggests using the section properties given on the Extended Span Precast Girders website. These details can be modified as follows:

I-Section: The top flange thickness can be increased up to 2 inches to accommodate more prestressing strands in the top flange.

U-Section: Web width of these sections may be varied to optimize the sections in meeting design requirements.

5.4.1.4 Design Criteria

Provide a minimum of two tendons per web.

Use diaphragms at all bearing locations.

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

When providing pre-tensioning in addition to post-tensioning, 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.
- Replace Restriction B with not more than 75% of the debonded strands, or 10 strands, whichever is greater, shall have the debonding terminated at any section.
- Replace Restriction C with longitudinal spacing of debonding termination locations shall be the larger of 36 inches or $60d_b$ apart, where d_b is the diameter of the strand.
- Do not design for Restriction E.

- For I-Sections, replace Restriction I with:
    - Bond strands placed within the horizontal limits of the web, when strands are located in the web.
    - Uniformly distribute debonded strands.
    - Bond the outer-most strand in each row.
- For U-Sections, replace Restriction J with:
    - Uniformly distribute debonded strands.
    - Bond the outer-most strand in each row.

Prestressed, precast sections must meet the following at release:

- Use the concrete release strength (f'_{ci}) for the following stress limitations:
    - Tensile stress < 0.24 $\lambda\sqrt{f'_{ci}}$ (ksi).
    - Compressive stress < $0.65f'_{ci}$ (ksi).
- Do not drape pretensioning strands. Debond the strands as needed.
- Strand stress after the seating of chucks is limited to $0.75f_{pu}$ for low-relaxation strands.
- Use an effective strand stress after release of $0.75f_{pu} - \Delta f_{pES}$

The precast sections must meet the following requirements for transportation:

- Prestressed Sections:
    - Factor the self-weight load by 1.33.
    - Use the concrete release strength (f'_{ci}) for the following stress limitations:
        - Tensile stress < 0.24 $\lambda\sqrt{f'_{ci}}$ (ksi).
        - Compressive stress < $0.65f'_{ci}$ (ksi).
- Non-Prestressed Sections:
    - Factor the self-weight load by 1.33.

Design the section as a reinforced concrete member, subject to the provisions in <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).
- Tendon stress before anchor set is limited to the lesser of $0.77f_{pu}$ and the stress limits in <Article 5.9.2.2> for low-relaxation strands.
- Use the final concrete strength (f'_c) for the following stress limitations:

  Tensile stress < $0.24\,\lambda\sqrt{f'_c}$ (ksi).

  Compressive stress < $0.6f'_c$ (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 calculated by hand as outlined in Section 5.3.1 or by analysis software that has concrete time dependent capabilities to capture the effect of creep and shrinkage.
- Compressive stress limitations:

  Service I Loading < $0.6f'_c$.

  Effective Prestressing and Permanent (Dead) Loading < $0.45f'_c$

  Fatigue I live loads plus one-half of the sum of stresses due to prestress and permanent (dead) loads < $0.40f'_c$.
- 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 limitations:

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

Evaluate principle tensile stresses using section properties that account for the presence of post-tensioning ducts in their ungrouted and grouted conditions.

All post tensioning must be done prior to placement of the deck. Post-tensioning after the deck is placed is permitted if a viable re-decking strategy is provided.

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-inch 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>. Use the General Procedure as provided by <Article 5.7.3.4.2>. Do not use the provisions of <Appendix B5>. When the effective web width must be reduced, reduce it by 25% of the outer diameter of the splice coupler for grouted ducts. Apply a shear strength reduction factor for the presence of grouted post-tensioning ducts as outlined in <Article 5.7.3.3>.

Only apply the requirement in <Article 5.7.3.5> from inside face of support to inside face of support. Do not calculate from the inside face of support to the end of the beam.

Design ultimate moment based upon Strength I loading for the final condition.

Refer to Section 5.3.1 for interface shear design of the deck to girder flange interface.

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.

Include 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.
- Final girder elevation points.
- Post tensioning sequence.

- Jacking stresses for prestressing strand and post-tensioned tendons.

Require contractors to provide a temporary bracing plan for the girders.

Require contractors to provide shoring and erection plan.

Commentary

A calculated positive (upward) camber is desired after application of all permanent (dead) loads for segments resisting positive flexural moment. This may not be possible for all roadway geometries.

Full depth diaphragms at the splices are typically cast continuously with the splices; this helps eliminate honeycombing that may occur at the splices.

5.4.1.5 Detailing

Provide 1.5-inch clear cover to reinforcing steel for precast elements (i.e. girders) and 2-inch clear cover for cast-in-place elements (i.e. diaphragms).

Provide a minimum 5-foot tangent length of tendon from anchorage bearing plate to initial point of curvature. Do not exceed the minimum radius of curvature for ducts based on published values from approved vendors.

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> for the post-tensioning forces and <Article 5.9.4.4> for the pretensioning forces. To determine the total required reinforcing for the anchorage zone, combine the required reinforcing for both the post-tensioned anchorage zone and the pretensioned anchorage zone. Provide anchor zone reinforcing at each debonded section, for the strands terminated there.

5.4.2 Segmental Spans

5.4.2.1 Materials

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

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

Use prestressing steel, duct, and cementitious grout as specified in Item 426, "*Post-Tensioning*".

5.4.2.2 Geometric Constraints

Use minimum cross-section dimensions as specified in <Article 5.12.5.3.11>.

Use a web spacing not greater than 32.0 feet measured between interior faces of exterior webs and centerlines of interior webs. Use a cantilever overhang length of the top flange less than 0.60 times the interior clear span of the top flange. When the cantilever overhang exceeds 0.45 times the interior clear span, investigate the top flange deflections and geometry control during casting and erection.

Where the clear span of the top slab between web faces exceeds 15.0 feet or the deck cantilever overhang exceeds 0.45 times the clear span between web faces, transversely prestress the top slab over the entire deck width.

For maintenance and inspection purposes, use a clear height of interior box sections not less than 6 feet.

5.4.2.3 Structural Analysis

Analyze segmental spans in accordance with <Article 4.6.2.9>. Analysis must account for time-dependent construction methods; short- and long-term post-tensioning, creep, and shrinkage effects; and secondary force effects due to post-tensioning.

Estimate shrinkage and creep using the provisions of the International Federation of Structural Concrete (*fib*) Model Code for Concrete Structures (*fib* 2010 or *fib* 2020), or the Euro-International Committee for Concrete/ International Federation for Prestressing (CEB-FIP) Model Code 1990 (CEB 1990); include the effects of specific materials (when known), structural dimensions, site conditions, construction sequence, and concrete age at various stages of erection as shown in the contract documents.

Perform transverse analysis of segmental box girder cross-sections in accordance with <Article 4.6.2.9.4> or using refined finite element modeling (FEM) methods.

Include the following in the plans:

- construction method and schedule used as the basis of design.
- assumed equipment reactions on the permanent structure and temporary supports required during erection.
- the specific code used for calculating prestress losses and cambers.
- key parameters assumed for creep, shrinkage, relaxation and relative humidity.

5.4.2.4 Design Criteria

Check limit states using the following load combinations:

- Longitudinal design for flexure: Service I (concrete compression), Service III (concrete tension), Strength I, Strength III and Strength IV.
- Longitudinal design for shear and torsion: Strength I, Strength III and Strength IV.
- Principal Tension check in webs: Service III.

- Transverse design of web, flanges and overhangs: Service I and Strength I.

In addition to the limit states and loads specified in Chapter 3 and outlined above, apply the construction loads and combinations in <Article 5.12.5.3.2> through <Article 5.12.5.3.4>. 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 <Article 5.9.2.2> and <Article 5.9.2.3>, except limit the concrete compressive stress for temporary stresses before losses to $\leq 0.65 f'_{ci}$. Use limits for severe corrosive conditions in areas of the state where de-icing agents are frequently used or in marine environments.

Calculate prestress losses for creep, shrinkage, elastic shortening, and relaxation by analysis software that has concrete time-dependent capabilities to capture the effect of creep and shrinkage.

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>. Account for the cumulative effect of shear, torsion, and torsion and transverse bending in the webs of box girders when determining the total reinforcement demand. <Equations C5.12.5.3.8d-1> and <Equation C5.12.5.3.8d-2> may be used for this purpose unless maximum shear and torsion effects are likely to be concurrent with maximum transverse bending.

Determine tie back reinforcement behind anchorages as required in <Article 5.9.5.6.7b>. For cast-in-place construction, 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.

Provide provisional post-tensioning force for future load, as described in <Article 5.12.5.3.9c>. Any extra design resistance in the original design cannot be used to meet these requirements.

5.4.2.5 Detailing

Provide 2 inches of clear cover to reinforcing steel for entire cross section. Add 1 inch grinding allowance to top slab. Increase top slab clear cover to 2.5 inches for areas of the state where deicing agents are frequently used.

Include shear keys in webs of precast and cast-in-place segmental spans. For cast-in-place segmental spans, space shear keys a minimum of 1 foot-6 inches along centerline of web. For precast segmental spans, provide shear keys either spaced a minimum of 1 foot-6 inches along centerline of web or consistent with <Article 5.12.5.4.2>.

Provide a minimum 5-foot tangent length of tendon from anchorage bearing plate to initial point of curvature. Do not exceed the minimum radius of curvature for ducts based on published values from approved vendors.

Design longitudinal reinforcement above a horizontal construction joint for restraint shrinkage caused by the casting sequence.

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 planned for and detailed in the plans.

Provide a ¾ inch deep continuous drip-bead adjacent to deck coping as shown in the Miscellaneous Slab Details for Prestressed Concrete I-Girders (IGMS) standard drawing.

The following are requirements for access openings:

- Provide at least two openings per box girder line or cell.
- Space openings no more than 600 feet on center.
- Every part of the box girder shall be no further than 300 feet from the center of the closest opening.
- Provide openings that are a minimum of 36 inches x 42 inches or 36 inches in diameter at a minimum.
- Do not place access openings where structural performance or secure access is compromised.
- Avoid placing access openings over traffic lanes, sloped embankment, standing water or other locations that reduce safe access for inspection personnel.
- Place openings through the top flange as far to one side as practical to reduce lane closures during inspection.
- Secure all access openings with a door, hatch, or manhole cover that can be secured.
- Use watertight covers for access holes in the top flange.

The following are requirements for openings through interior diaphragms and webs:

- Provide openings between cells of a multi box superstructure at the pier tables and through all interior diaphragms.
- Provide openings that are a minimum of 36 inches wide and 42 inches tall. Make the height of openings through diaphragms and webs as tall as practical to provide easy access by construction workers and inspection personnel. 6 feet is the desirable opening height.

- Keep openings free of attachments, objects, and protrusions. Place conduit and utilities through independent openings. Detail utility and conduit attachments to allow the structure to move under thermal effects without damaging the utility or conduit.

Provide drainage at the low points of the bottom flange. Provide drainage uphill of any obstruction where water may pond. Use scuppers that extend a minimum of 2 inches below the bottom soffit when draining through the bottom flange.

Provide illumination inside each cell.

Commentary

Access openings through the end diaphragms or the bottom flange are preferred if site conditions allow for it. Access openings through the top flange are discouraged and should only be used if the other options are not feasible.

5.5 Cast-in-Place Concrete Superstructures

This section documents policy on the design of cast-in-place superstructures.

5.5.1 Cast-in-Place Concrete Slab Spans

5.5.1.1 Materials

Use normal weight 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, "Reinforcement for Concrete"):

- Epoxy-coated reinforcing steel meeting the requirements of ASTM A775 or A934.
- Galvanized reinforcing steel.
- Dual coated reinforcing steel meeting the requirements of ASTM A1055.
- Low-carbon and low-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.

5.5.1.2 Geometric Constraints

Cast in place concrete slab spans are defined by the roadway geometry and should be set with the same points that define the cross slope on the bridge. Slabs are a constant thickness, and the cap is sloped to accommodate the cross-slope of the slab.

The maximum skew angle for slab span bridges is 30°. With skewed spans, use shear keys that are 2 inches deep by 6 feet 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 feet 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 inches. Use 1.25 inches bottom clear cover.

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

5.5.1.3 Structural Analysis

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

Design using 1-foot-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 feet.

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 <Article 5.12.2.1> and <Article 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>.

5.5.1.4 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 account for 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 No. 4 reinforcing bars at 12 inches 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.

5.6 Steel Beams and Girders

This section documents policy on the design of steel superstructures.

For additional recommendations, refer to the *TxDOT Preferred Practices for Steel Bridge Design, Fabrication, and Erection.*

5.6.1 Straight Plate Girders

5.6.1.1 Materials

Use ASTM A709 Grade 50W steel for unpainted bridges. Use ASTM A709 Grade 50 steel for painted bridges. Use ASMT A709 Grade HPS 70W steel for unpainted and painted bridges if it is economical or otherwise beneficial to do so.

Use 0.875 inches or 1 inch diameter bolts for bolted connections.

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

5.6.1.2 Geometric Constraints

Minimum flange width is 0.20D, where D is web depth, but not less than 15 inches.

Minimum flange thickness is 0.75 inches.

Minimum web thickness is 0.50 inches.

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

Minimum split-pipe (half-round) stiffener thickness is 0.625 inches for rolled plate and 0.50 inches for round hallow structural sections (HSS).

Satisfy the span-to-depth ratios in <Article 2.5.2.6.3> as a minimum, unless vertical clearance constraints demand a shallower superstructure.

5.6.1.3 Structural Analysis

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.

5.6.1.4 Design Criteria

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 feet if all limit state 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 <Article 6.8.4> and <Article 6.9.3>.
- Diaphragm and cross-frame designs may, but are not required to, meet the provisions of <Article 6.7.4.2.2. If <Article 6.7.4.2.2> is used, apply the following modifications:

  Use the factored major-axis bending moment (M_u) at each bracing line to determine the required stiffness ($(\beta_T)_{req}$) on a per-line basis. Do not use the maximum moment within a given (positive or negative) bending region.

  When calculating the actual stiffness ($(\beta_T)_{act}$) at support lines, use an in-plane girder stiffness (β_g) equal to infinity.

Lean-on bracing design, as described in *Refined Design Methods for Lean-On Bracing* (Bjelland et al) is permissible. For structures utilizing lean-on bracing systems, detail the assumed construction sequence in the plans. Refer to Appendix D for lean-on bracing design guidance.

Use composite design and place shear connectors along the full girder length.

Do not use longitudinal stiffeners unless web depth exceeds 120 inches.

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>, where f_r is the modulus of rupture.

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 feet.

Include an assumed stay-in-place formwork weight of 15 pounds per square feet (psf) in design.

Investigate and verify feasibility of a possible erection sequence during design and verify possible locations of shore towers and cranes. Consider traffic phasing with underlying roadways when considering locations of shore towers and cranes. Do not include detailed erection plans in plan set.

Specify continuous placement of bridge deck where possible, and staged placement only if required. Do not disallow continuous placement solely based on whether a continuous pour may be unfeasible for a contractor. 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 staged placement if staged placement is the only placement option. If both staged and continuous placement are given as options, base dead load deflection and camber on continuous placement as long as there is no significant difference in final camber and deflection between the two methods. State in the plans which placement option is assumed for the dead load deflection and camber. Provide a staged placement diagram indicating the intended pour sequence in the design if staged placement is specified. In the plans, state that for continuous placement, the minimum rate of placing and finishing shall not be less than that specified in Item 422, “Concrete Superstructures”.

For stud connector designs, minimum longitudinal stud connector spacing is limited to 4d, where d is the stud connector diameter. Do not exceed a stud connector spacing of 24 inches regardless of girder depth.

For dapped girder ends, utilize <Article D6.5.2> to avoid the use of additional stiffeners at dap bend points per <Article 6.10.1.4>.

In lieu of permanent bottom flange lateral bracing, increase bottom flange size if practical. If considering the use of bottom flange lateral bracing, contact the Bridge Division (BRG_Design@txdot.gov) for approval.

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 ASTM F3125 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 inches, and preferably 0.25 inches, 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 inches. Instead, reduce bolt shear strength with <Equation 6.13.6.1.4-1>.

5.6.1.5 References

Bjelland, A.; C. Gasser, D. Fish, T. Helwig, M. Yarnold, S. Park, S. Patel, S. Hurlebaus, M. Hebdon, M. Engelhardt, and E. Williamson, “Refined Design

Methods for Lean-On Bracing," Report No. FHWA/ TX-24/0-7093-1, Center for Transportation Research, The University of Texas at Austin, Austin, TX. and Texas Transportation Institute, Texas A&M Transportation Institute, Bryan, TX. 2026.

Commentary

Resources:

Guidelines to Design for Constructability and Fabrication (G12.1-2020). American Association of State Highway and Transportation Officials, and National Steel Bridge Alliance. (https://www.aisc.org/globalassets/nsba/aashto-nsba-collab-docs/nsbagdc-4.pdf)

5.6.2 Curved Plate Girders

5.6.2.1 Materials

Use ASTM A709 Grade 50W steel for unpainted bridges. Use ASTM A709 Grade 50 steel for painted bridges. Use ASTM A709 Grade HPS 70W steel for unpainted and painted bridges if it is economical or otherwise beneficial to do so.

Use 0.875 inches or 1 inch diameter bolts for bolted connections.

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

5.6.2.2 Geometric Constraints

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

Minimum flange thickness is 1 inch.

Minimum web thickness is 0.50 inches.

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

Minimum split-pipe (half-round) stiffener thickness is 0.625 inches for rolled plate and 0.50 inches for round HSS.

Satisfy the span-to-depth ratios in <Article 2.5.2.6.3> as a minimum, unless vertical clearance constraints demand a shallower superstructure.

5.6.2.3 Structural Analysis

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.

5.6.2.4 Design Criteria

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:

- Diaphragm and cross-frame members are primary members. Verify their adequacy for the Strength Limit State and other applicable limit states.
- The maximum spacing is 20 feet 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 criteria provided in <Article 6.8.4> and <Article 6.9.3>.
- Diaphragm and cross-frame designs may, but are not required to, meet the provisions of <Article 6.7.4.2.2>. If <Article 6.7.4.2.2> is used, apply the following modifications:
    - Use the factored major-axis bending moment (M_u) at each bracing line to determine the required stiffness ($(\beta_T)_{req}$) on a per-line basis. Do not use the maximum moment within a given (positive or negative) bending region.
    - When calculating the actual stiffness ($(\beta_T)_{act}$) at support lines, use an in-plane girder stiffness (β_g) equal to infinity.

Use composite design and place shear connectors along the full girder length.

Do not use longitudinal stiffeners unless web depth exceeds 120 inches.

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 feet.

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

Investigate a possible erection sequence during design and verify possible locations of shore towers and cranes. Consider traffic phasing with underlying roadways when considering locations of shore towers and cranes. Do not include detailed erection plans in plan set.

Specify continuous placement of bridge deck where possible, and staged placement only if required. Do not disallow continuous placement solely based on whether a continuous pour may be unfeasible for a contractor. 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 staged placement if staged placement is the only placement option. If both staged and continuous placement are given as options, base dead load deflection and camber on continuous placement if there is no significant difference in final camber and deflection between the two methods. State in the plans which placement option is assumed for the dead load deflection and camber. Provide a staged placement diagram indicating the intended pour sequence in the design if staged placement is specified. In the plans state that for continuous placement, the minimum rate of placing and finishing shall not be less than that specified in Item 422, "Concrete Superstructures".

For stud connector designs, minimum longitudinal stud connector spacing is limited to 4d, where d is the stud connector diameter. Do not exceed a stud connector spacing of 24 inches regardless of girder depth.

For dapped girder ends, utilize <Article D6.5.2> to avoid the use of additional stiffeners at dap bend points per <Article 6.10.1.4>.

In lieu of permanent bottom flange lateral bracing, increase bottom flange size if practical. If considering the use of bottom flange lateral bracing, contact Bridge Division (BRG_Design@txdot.gov) for approval.

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 ASTM F3125 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 inches, and preferably 0.25 inches, 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 inches. Instead, reduce bolt shear strength with <Equation 6.13.6.1.4-1>.

5.6.3 Trapezoidal Box Girders

The use of steel tub girders requires Bridge Division approval.

Steel tub girders may be allowed when steel I plate-girders are not a viable option for the project.

5.6.3.1 Materials

Use ASMT A709 Grade 50W steel for unpainted bridges. Use ASTM A709 Grade 50 steel for painted bridges. Use ASTM A709 Grade HPS 70W steel for unpainted and painted bridges if it is economical or otherwise beneficial to do so.

Use 0.875 inches or 1 inch diameter bolts for bolted connections.

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

5.6.3.2 Geometric Constraints

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

Minimum flange thickness is 1 inch.

Minimum web thickness is 0.50 inches.

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

Minimum split-pipe (half-round) stiffener thickness is 0.625 inches for rolled plate and 0.50 inches for round HSS.

Satisfy the span-to-depth ratios in <Article 2.5.2.6.3> as a minimum, unless vertical clearance constraints demand a shallower superstructure.

5.6.3.3 Structural Analysis

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.

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 4.6.1.2.4c> 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.

5.6.3.4 Design Criteria

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

Use composite design and place shear connectors along the full girder length.

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 feet.

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

Investigate a possible erection sequence during design and verify possible locations of shore towers and cranes. Consider traffic phasing with underlying roadways when considering locations of shore towers and cranes. Do not include detailed erection plans in plan set.

Specify continuous placement of bridge deck where possible, and staged placement only if required. Do not disallow continuous placement solely based on whether a continuous pour may be unfeasible for a contractor. 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 staged placement if staged placement is the only placement option. If both staged and continuous placement are given as options, base dead load deflection and camber on continuous placement if there is no significant difference in final camber and deflection between the two methods. State in the plans which placement option is assumed for the dead load deflection and camber. Provide a staged placement diagram indicating the intended pour sequence in the design if staged placement is specified. In the plans state that for continuous placement, the minimum rate of placing and finishing shall not be less than that specified in Item 422, "Concrete Superstructures".

For stud connector designs, minimum longitudinal stud connector spacing is limited to 4d, where d is the stud connector diameter. Do not exceed a stud connector spacing of 24 inches regardless of girder depth.

For dapped girder ends, utilize <Article D6.5.2> to avoid the use of additional stiffeners at dap bend points per <Article 6.10.1.4>.

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 ASTM F3125 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 inches, and preferably 0.25 inches, 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 inches. Instead, reduce bolt shear strength with <Equation 6.13.6.1.4-1>.

5.6.3.5 System Redundancy Evaluation for Steel Twin Tub Girders

5.6.3.5.1 Overview

This section presents an LRFD based method to evaluate the system redundancy of spans with twin tub-girder cross-sections. The method allows the bottom tension flanges and webs attached to those flanges in the span under consideration to be designated as System Redundant Members (SRMs), rather than Nonredundant Steel Tension Members (NSTMs), previously known as Fracture Critical Members (FCMs). The methodology evaluates system redundancy of spans with two tub (i.e. twin tub) girders in the cross section such that the span will not collapse after the fracture of one girder. The methodology establishes a simplified method and refined analysis method for evaluating system redundancy, validated through behavioral observations obtained from a series of full-scale tests (Barnard et al., 2010).

Commentary

The probability of a fracture for tub girders designed for infinite fatigue life is considered exceedingly small in comparison to the bridge's design life. Therefore, the method addresses the design of a simulated fracture with Extreme Event III limit state.

Modeling the Response of Fracture Critical Steel Box-Girder Bridges (Barnard et al., 2010), demonstrated that spans with twin tub-girder cross-sections can possess adequate system redundancy to prevent collapse and carry a substantial live load in excess of HL-93.

5.6.3.5.2 Structural Analysis

All two tub girder bridges must satisfy the requirements in this manual and must be evaluated for system redundancy of critical spans at Extreme Event III limit state as described in Section 3.8. Two types of analysis, approximate and refined, can be used to evaluate the Extreme Event III.

Approximate structural analysis:

Approximate structural analysis, as described in *Modeling the Response of Fracture Critical Steel Box-Girder Bridges* (Barnard et al., 2010) and the simplified method for two tub girder bridges is permitted when:

- Critical Spans do not exceed 250 feet.
- Supports are skewed no more than 20 degrees.
- Horizontal curvature is greater than 700 feet.
- Engineer ascertains that the use of an approximate analysis method is adequate.

If the criteria above are met, the simplified method for analyzing twin tub girder bridges in the event of a fracture presented in Barnard et al. (2010) is permitted. Refer to Appendix C: Steel Twin Tub Girder System Redundancy Simplified Method Guide for guidance on performing the simplified method.

For the simplified method redundancy evaluation, the bridge under consideration needs to satisfy three conditions:

1. Intact girder has adequate shear and moment capacity.
2. Deck has adequate shear capacity.
3. Shear studs connections have adequate tension capacity.

If the twin tub girder bridge satisfies the first two conditions: 1) the intact girder having adequate shear and moment capacity and 2) adequate shear capacity of the deck; but doesn’t satisfy 3) the condition of adequate tension capacity in the shear studs, then a more refined analysis can be used to evaluate the ability of the deck to transmit load to the intact girder without the shear studs connecting the deck to the fractured girder.

For the simplified method 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.

<u>Refined structural analysis:</u>

Refined structural analysis as described in *Modeling the Response of Fracture Critical Steel Box-Girder Bridges*, (Barnard et al., 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 this Section.

When a structure is subject to Extreme Event III load combination in the faulted state, performance of the structure is evaluated for structural integrity and capacity to withstand the loading conditions.

Performance requirements for the faulted bridge for refined structural analysis method:

- Displacement at Supports: The maximum horizontal/lateral displacement is 50% of the sole plate width or bearing contact area.
- Vertical deflection: Dead Load deflection post fracture does not exceed L/50, where L is the span length.
- Stabilization: System must attain equilibrium after fracture. This can be demonstrated by showing the time dependent variations of both girder deflections and support reactions exhibit substantial stability by converging to a constant value.

A structurally continuous railing, barrier, or median, acting compositely with the supporting components, may be considered to be structurally active at Extreme Event III limit state when evaluating system redundancy.

TxDOT and the Engineer shall agree upon the location of the assumed fracture, degree to which dynamic effects associated with a fracture are included in the analysis, fineness of models, and choice of element type structural analysis approach.

TxDOT and the Engineer shall mutually agree upon the software used for the analysis taking into consideration software's ability to adequately capture the complexity of the analysis.

For Extreme Event III limit state for twin tub girder bridges, investigation for system redundancy shall be limited to critical end spans of continuous units and all simple spans.

One girder within the critical 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.

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.

Commentary

If a bridge meeting the criteria mentioned above for the simplified method does not satisfy the strength checks using the results from the simplified analysis method, the designer can either revise their design to satisfy the strength requirements, or a three-dimensional finite element model can be developed to provide a more accurate estimate of the bridge performance in the event of fracture. Finite element modeling techniques for assessing system redundancy are also described in Barnard et al. (2010).

One of the most accurate ways to assess the redundancy of complex systems, such as twin tub-girder systems, is through finite-element modeling (Samaras et al., 2012). Such models, however, require a substantial amount of time to develop and analyze.

The 1.10 live load factor in Extreme Event III limit state in Section 3.8, is considered appropriate for determination of system redundancy as specified elsewhere in this section, in consideration of the very low probability of fracture of one steel tub girder in a twin tub-girder superstructure cross-section which has been designed for infinite fatigue life.

If a future lane configuration is known at the time of design, the future lane configuration should also be considered when evaluating system redundancy. It is considered overly conservative to place additional live load in a striped shoulder to represent a parked or disabled vehicle when evaluating system redundancy.

5.6.3.5.3 Design Criteria

These provisions 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, use the applicable Extreme Event III load combination specified in the supplement of Table 3.4.1-1 in Section 3.8.

Twin tub-girder spans satisfying the system redundancy requirements of this Section 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 nonredundant steel tension members (NSTM) when evaluated based on load path redundancy alone, shall instead be designated in the contract documents as system redundant members (SRMs) and the frequency for the hands-on in-service inspection shall be in accordance with the protocol for NSTM as described in TxDOT Bridge Inspection Manual, Chapter 4, Section 7. The SRMs shall be fabricated according to Clause 12 of the ASSTHO/American Welding Society (AWS) D1.5M/D1.5 *Bridge Welding Code* Fracture Control Plan (FCP).

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. Design these diaphragms to act compositely with the slab. With the shear connectors designed as specified in this section under the subsection Shear: Shear Connectors below.

Additionally, at least two permanent full depth 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 full depth 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 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.

External intermediate bracing elements are sometimes removed after the deck placement for aesthetic purposes but must permanently remain in the structure in this case to provide additional load paths in the event of a fracture.

Commentary

External intermediate diaphragms or cross-frames can help in some instances to obtain a reasonably uniform concrete deck thickness during the deck placement by controlling the box-girder twist relative to the adjacent box girders. Helwig et al. (2007) and Helwig et al. (2004) have shown that 2 to 4 external intermediate diaphragms or cross-frames are typically sufficient for this purpose in horizontally curved box-girder spans.

The requirement to provide permanent external intermediate diaphragms in spans of twin tub-girder cross-sections designed for Extreme Event III limit state as specified elsewhere in this section is intended to enhance system redundancy by providing additional load paths on each side of an assumed fracture location within the span under consideration.

<u>Connections:</u>

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

<u>Flexure:</u>

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 <Article 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>. Saint 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.
- **Intact girder** - The shear in the intact girder due to the torsion and vertical loads transferred from fractured girder should be included in the strength check.
- **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., 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. The maximum longitudinal shear stud spacing shall not exceed 16.0 inches.

- **Top Flange Lateral Bracing** - Top flange lateral bracing can be considered part of the resisting section for Saint Venant torsional shears in addition to the concrete deck. The contributions of the deck and top lateral bracing are additive.
- **Torsion** - The radius of curvature must be taken into account for the intact tub girder. A decrease in the radius of curvature increases the torsion on the bridge, which must be resisted by the intact girder in the event of a fracture of a critical tension flange. Under such conditions, the eccentricity should not be taken as the radial distance between the centerlines of the girders and the loads; it should be computed as the distance from the center of gravity of the loads to the line of the intact girder interior supports. The center of gravity for non-prismatic girders can be determined by using equations developed by Stith et al. (2010), modified for the case of tub girders.

  This applied torque is resisted by a couple generated by the bearings of the two girders (i.e., bearing reactions). The reaction at the bearing of the fractured girder is equal to the torque applied to the intact girder divided by the distance between the bearings of the two girders. If two bearings per girder are used, the torque applied to the intact girder could be distributed to its two bearings.

Commentary

The flexural resistance of the intact girder in regions of positive and negative flexure needs to be checked after the assumed fracture event to ensure that the girder can sustain the load transferred from the fractured girder in conjunction with the self-weight of the intact composite girder.

Results from the test program described in Barnard et al. (2010) on the full-scale test bridge indicated that the torsion introduced into the intact girder was nearly symmetrical, indicating that the torque was resisted equally at each end of the intact girder. Therefore, for simplicity, it was assumed that the intact girder had symmetrical torsional boundary conditions so that each end resisted one-half of the total applied torque. For the simplified method, boundary conditions would be idealized or approximated, so the research recommends symmetrical torsional boundary conditions. Due to the fracture location or other mitigating factors, this simplification may not be appropriate in all cases, and an engineer may have to approximate the boundary conditions for torsion or complete a more detailed analysis. The research presents equations for calculating torques for dead and live load. These equations were used to calculate the torques on the experimentally tested bridge and compared them with torques computed using strain gauge data. The results showed that the proposed simplified method to compute the applied torque is conservative because it overestimated the applied torque. Furthermore, it is assumed that the live load and dead load is uniformly distributed.

5.6.3.5.4 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 feet 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 with a minimum height of at least 32 inches shall be provided and should be considered to be structurally active for the analysis at the Extreme Event III limit state as permitted in this section.

5.6.3.5.5 Submittal and Approval

To satisfy FHWA requirements, TxDOT Bridge Division must approve each steel twin tub girder bridge design for system redundancy. At 60% plan submittal, 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 shall meet the requirements of <Chapter 6>. The package shall include the calculations demonstrating redundancy and an explanation of assumptions for modeling and modeling method used if a refined method is chosen.

Upon TxDOT's acceptance and approval of 100% Plans, submit final calculation package in accordance with Section 2.5 and Section 2.6, including final redundancy calculations, as well as the full completed refined analysis records/computer models as this information will be retained and included in the bridge inspection management system.

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

5.6.3.5.6 References

Barnard, T.; C. Hovell, J. Sutton, J. Mouras, B. Neumann, V. Samaras, J. Kim, E. Williamson, and K. Frank, "Modeling the Response of Fracture Critical Steel Box-Girder Bridges", FHWA/TX-10/9-5498-1, Center for Transportation Research, The University of Texas at Austin, Austin, TX., (https://library.ctr.utexas.edu/ctr-publications/9-5498-1.pdf), 2010.

Mouras, J., J. Sutton, K. Frank, and E. Williamson, "The Tensile Capacity of Welded Shear Studs", FHWA/TX-09/9-5498-2, Center for Transportation Research, The University of Texas at Austin, Austin, TX., (https://library.ctr.utexas.edu/ctr-publications/9-5498-2.pdf), 2008.

Helwig, T., J. Yura, R. Herman, E. Williamson, and D. Li, "Design Guidelines for Steel Trapezoidal Box Girder Systems", FHWA/TX-07/0-4307-1, Center for Transportation Research, The University of Texas at Austin, Austin, TX., (https://library.ctr.utexas.edu/ctr-publications/0-4307-1.pdf) 2007.

Helwig, T.A., R.S. Herman, and D. Li. "Behavior of Trapezoidal Box Girders with Skewed Supports", 0-4148-1. University of Houston, Houston, TX., (https://library.ctr.utexas.edu/digitized/texasarchive/phase1/4148-1-uh.pdf), 2004.

Samaras, V. A., J. P. Sutton, E. B. Williamson, and K. Frank. "*Simplified Method for Evaluating the Redundancy of Twin Steel Box-Girder Bridges*" Journal of Bridge Engineering. American Society of Civil Engineers, Reston, VA, Vol. 17, No. 3, May/June, pp. 470-480. 2012.

Stith, J., A. Schuh, J. Farris, B. Petruzzi, T. Helwig, E. Williamson, K. Frank, M. Engelhardt, and H. J. Kim, "Guidance for Erection and Construction of Curved I-Girder Bridges", Technical Report FHWA/TX-10/0-5574-1, Center for Transportation Research, The University of Texas at Austin, Austin, TX., (https://library.ctr.utexas.edu/ctr-publications/0-5574-1.pdf) 2010.

Chapter 6 Substructure Design

6.1 Overview

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

6.2 Limit States and Loads for Substructures

TxDOT recommends the following limit states for design of bridge system substructure components <Article 3.4.1>:

Table 6.2-1 – Recommended Limit States

Component	Limit State
Concrete Bent Caps	Strength I, Service I, and Service I with Dead Load only (1)
Columns	Strength I, III, and V, Service I and IV, and Extreme II (for vehicle or vessel collision, when required)

Traditionally, non-integral bent caps only consider Dead Load (DC and DW), and Live Load (LL). Braking (BR), Wind (WS and WL), and Temperature (TU and TG) are typically neglected. For fully integral bent caps, braking, wind, and temperature should be considered.

6.3 Foundations

This section documents policy on foundations.

6.3.1 Materials

Consider the exposure environment in the selection of foundation types, especially in waterway environments. Use of steel piling must incorporate recommendations in the TxDOT Guidelines for the Use of Steel Piling for Bridge Foundations. Additional durability guidance of <Article 10.7.5> applies.

Do not use timber piling without obtaining approval from the TxDOT Bridge Division.

6.3.2 Design Criteria

Evaluate geotechnical resistance in accordance with the *TxDOT Geotechnical Manual-LRFD*.

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 the *TxDOT Scour Evaluation Guide* and the *TxDOT Scour Analysis Guide* for additional guidance on design for scour.

For monoshaft foundation framing with single column bents, perform analysis to examine the consequences of deflection under lateral loads.

6.3.3 Detailing

Where lap splices are not possible, see Section 4.11 for non-contact lap splice requirements.

For drilled shafts greater than 4 feet in diameter, consider using a concrete cover to reinforcing steel of 6 inches.

6.4 Footings

This section documents policy on footings.

6.4.1 Materials

Use normal weight concrete Class C concrete (f_c' = 3.6 ksi). Higher concrete strengths may be required in special cases. If higher concrete strengths are required, use Class F, H, or K. Mass concrete may be needed depending on the size of the cap.

Use Grade 60 reinforcing steel. Higher reinforcing steel grades may be used provided their use satisfies requirements in AASHTO.

6.4.2 Design Criteria

Design footings using the strut-and-tie modeling provisions of Section 4.13 and <Article 5.8.2>.

6.5 Columns

This section documents policy on columns used for multi-column bents and for single column bents.

6.5.1 Columns for Multi-column Bents

6.5.1.1 Materials

Use normal weight concrete Class C concrete ($f_c' = 3.6$ ksi). Higher concrete strengths may be required in special cases. If higher concrete strengths are required, use Class F, H, or K. Mass concrete may be needed depending on the size of the column.

Use Grade 60 reinforcing steel. Higher reinforcing steel grades may be used provided their use satisfies requirements in AASHTO.

6.5.1.2 Geometric Constraints

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

6.5.1.3 Structural Analysis

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

- Column spacing does not exceed 16 feet.
- Columns meet the following minimum diameters based on superstructure type:
    - For slab spans, 24-inch diameter for stream crossings and 36-inch diameter for grade crossings.

For slab beam and simply supported steel beams spans, 30-inch diameter for stream crossings and 36-inch diameter for grade crossings.

For box beam and spread box beam spans, 36-inch diameter.

For Types Tx28 through Tx54 TxGirder spans, 36-inch diameter.

For Types Tx62 and Tx70 TxGirder spans, 42-inch diameter.

- Columns less than the following maximum heights based on column diameter:

  24-inch diameter with a max height of 20 feet.

  30-inch diameter with a max height of 30 feet.

  36-inch diameter with a max height of 36 feet.

  42-inch diameter with a max height of 42 feet.

  These heights are measured from the bottom of the cap to the top of the drilled shaft or footing. For drilled shaft foundations in stream crossings, the bottom of the column is to be taken at the bottom of the scour envelope.
- Columns reinforced with #4 spirals with a 3-inch pitch.
- Columns reinforced vertically with at least 1% reinforcement.

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

Round columns with diameters less than 36-inch are unable to provide sufficient structural resistance when subjected to collision loads.

Account for global stability of columns subject to second order effects. Ultimate moment (M_u) including secondary effects should not exceed 1.25M_u due to first-order effects.

For analysis, the designer should consider predicted scour when determining column heights.

Moments can be magnified to account for slenderness (P-delta) effects by using <Article 5.6.4.3> or other analytical methods. <Article 5.6.4.3> is typically considered highly conservative.

Column size may change within the bent height, producing a multi-tiered bent. Consider multi-columns bent tiers with web walls to be braced in the transverse direction. Column capacity in the longitudinal direction is not affected by the web wall.

For multi-tier bents with square or round columns separated by tie beams, analyze as a frame, and magnify transverse and longitudinal moments separately.

Design and model single tier bent columns without a tie beam or web wall as individual columns with bottom conditions fixed against rotation and deflection at the location of fixity. The top-of-column should be modeled consistently with the detailing of the cap to column connection.

In most cases, it can be assumed when determining fixity conditions for loads that columns on single drilled shafts are fixed at three shaft diameters but not less than 10 feet below the top of the shaft. This should always be reviewed by a Geotechnical Engineer to evaluate and determine the fixity condition for the project location.

Commentary

MacGregor and Hage (1977) found through analytical research that structures designed using second-order analysis maintain a consistent probability of stability when the ratio of secondary to primary moments is 1.10 or less. Once this ratio exceeds 1.25, the likelihood of failure increases sharply.

Reference:

Macgregor, James G., and Sven Egil. Hage., *Stability Analysis and Design of Concrete Frames*, Journal of the Structural Division 103 (1977): 1953-1970.

6.5.1.4 Design Criteria

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

Evaluate concrete tensile stress at Service I and Service IV for prestressed elements.

Design columns subject to the vehicular collision load based on Section 4.14.

Commentary

See the TxDOT Bent Protection Guide for additional guidance on designing for vehicular collision.

6.5.1.5 Detailing

Where lap splices are not possible, see Section 4.11 for non-contact lap splice requirements.

Round columns are preferred for multicolumn bents with rectangular caps and are used for most structures. Square or rectangular columns may be used.

6.5.2 Columns for Single Column Bents or Piers

6.5.2.1 Materials

Use normal weight concrete Class C concrete (f_c' = 3.6 ksi). Higher concrete strengths may be required in special cases. If higher concrete strengths are required, use Class F, H, or K. Mass concrete may be needed depending on the size of the column.

Use Grade 60 reinforcing steel. Higher reinforcing steel grades may be used provided their use satisfies requirements in AASHTO.

6.5.2.2 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.

Global stability of column subject to second order effects need to be accounted for. Ultimate moment (M_u) including secondary effects shall not exceed 1.1M_u due to first-order effects.

Longitudinal and transverse moments can be magnified separately for P-delta effects using the method in <Article 4.5.3.2.2> or with a second order analysis computer program.

Effective length factor may be taken as 2.0 in both directions unless restraints provided by the superstructure sufficiently limit secondary moments.

Commentary

MacGregor and Hage (1977) found through analytical research that structures designed using second-order analysis maintain a consistent probability of stability when the ratio of secondary to primary moments is 1.10 or less. Once this ratio exceeds 1.25, the likelihood of failure increases sharply.

Reference:

Macgregor, James G., and Sven Egil. Hage., *Stability Analysis and Design of Concrete Frames*, Journal of the Structural Division 103 (1977): 1953-1970.

6.5.2.3 Design Criteria

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

Evaluate concrete tensile stress at Service I and Service IV for prestressed elements.

Design columns subject to the vehicular collision load based on Section 4.14.

Commentary

See the TxDOT Bent Protection Guide for additional guidance on designing for vehicular collision.

6.5.2.4 Detailing

Where lap splices are not possible, see Section 4.11 for non-contact lap splice requirements.

6.6 Bent Caps

This section documents policy on rectangular and inverted-tee reinforced bents caps, post-tensioned bent caps, and steel straddle caps.

6.6.1 Rectangular Reinforced Concrete Bent Caps

6.6.1.1 Materials

Use normal weight concrete Class C concrete (f_c' = 3.6 ksi). Higher concrete strengths may be required in special cases. If higher concrete strengths are required, use Class F, H, or K. Mass concrete may be needed depending on the size of the cap.

Use Grade 60 reinforcing steel. Higher reinforcing steel grades may be used provided their use satisfies requirements in AASHTO.

6.6.1.2 Geometric Constraints

Use a cap depth greater than the cap width, except as needed for the situations listed below:

- When widening a cap to meet the minimum support length per <Article 4.7.4.4>.
- When widening a cap to accommodate a cap to column connection when one or both elements are precast.
- Meeting the vertical clearance needs of a lower roadway.

Provide a cap width with a minimum of 3 inches wider than the supporting columns to allow column reinforcing to extend into the cap without bending.

For bents supporting slab beams and simply supported steel beams, use a width of at least 2.75 feet and 30-inch diameter drilled shafts as a minimum.

For bents supporting Tx28 through Tx54 girders, adjacent box beams, and X-beams use a width of at least 3.50 feet and 36-inch diameter drilled shafts as a minimum.

For bents supporting Tx62 and Tx70 girders, use a cap width of at least 4.00 feet and 42-inch diameter drilled shafts as a minimum.

6.6.1.3 Structural Analysis

In lieu of a more detailed analysis, it is permissible to analyze trestle pile and multi-column caps as simply supported beams on knife-edge supports at the center of piling or columns. If the column is wider than 4 feet, 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. Do not use PGSuper live load beam reactions.

Apply dead load reactions due to slab and beam weight as point loads at center of bearing. For all beams, distribute the weight of one railing, sidewalks, and/or medians to no more than three beams. Considerations should be given to wide medians and sidewalks, where the weight may be distributed to more than the prescribed number of beams. Distribute dead loads due to overlay, when used, evenly to all beams.

Commentary

If using CAP18, model the total live load reaction as two wheel loads, distributing the remainder of the live load over a 10-foot design lane width. Carefully consider lane boundaries to produce the maximum force effect at various critical locations:

$$w = \frac{LL_{Rxn} - 2\ x\ P}{10\ feet}$$

where:

w = The uniform load portion of the live load (kip/feet).

LL_{Rxn} = Live load reaction per lane = (LL_{Truck}*1.33)+LL_{Lane} ≥ 0.9(($LL_{TruckTrain}$*1.33)+LL_{Lane}) (kip/lane).

P = The load on one rear wheel of the HL-93 truck increased 33% for dynamic load allowance (kip). Typically, P = 16k * 1.33.

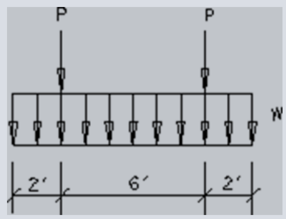


Figure C6.6.1.3-1: Recommended Live Load Model for CAP18

6.6.1.4 Design Criteria

Check limit states using Strength I.

Check 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 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 $0.6f_y$, where f_y is the yield strength of the reinforcing.

Check <Article 5.6.3.3> for minimum reinforcement.

When calculating spacing requirements per <Article 5.10.3.1> use the absolute maximum aggregate size, the smallest sieve with 100% passing, not the maximum nominal size of the aggregate per Item 421, “Hydraulic Cement Concrete”. If smaller reinforcing spacing is required, limit the sizes of aggregate in the material notes on the plan sheet. Contact Bridge Division for guidance.

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 feet wide or wider, take design negative moments at the effective face of the column.

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>.

Minimize the number of stirrup spacing changes.

When the exterior beam is 0.5d to 1.2d, where d is the shear depth, from the effective face of the column; flexural reinforcing extends more than 15 inches past the centerline of the beam; and with stirrups at 6-inch maximum spacing, there is sufficient development for bars up to #11 in size and at yield strengths up to 75 ksi. If the exterior beam is outside these bounds, provide development length based on <Article 5.10.8.2>.

When the exterior beam is closer than 1.2d, where d is the shear depth, from the effective face of the column, shear may be neglected in cantilever regions. Provide stirrups at 6-inch maximum spacing in the cantilever. This provision does not apply to hammerhead bents.

For typical multi-column bent caps supporting multiple beams, strut-and-tie modeling provisions of <Article 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 method.

Using <Appendix B5> to design shear for conventionally reinforced caps is the preferred method. <Article 5.7.3.4.2> may be used for prestressed caps.

If <Article 5.7.3.4.1> or <Article 5.7.3.4.2> is used, ensure that the minimum stirrup spacing requirements are not violated. If the required stirrup spacing is less than the minimum allowed per AASHTO, re-evaluate using <Appendix B5>, before increasing the shear depth.

Base lap lengths on tension lap requirements <Article 5.10.8.4.3>.

Design and reinforce bearing seat build-ups taller than 3 inches.

Commentary

Provide a design where all the tensile reinforcement yields at ultimate, which results in a ductile failure. For example, when there are 3 or more mats of reinforcement on the top or bottom of the bent cap, the reinforcement in the third or more mats will not yield. Bridge Division recommends using the strain compatibility design method to get the capacity of the bent cap.

Simplified procedure in <Article 5.7.3.4.1> for all designs and the general procedure in <Article 5.7.3.4.2> for nonprestressed caps result in overly conservative shear designs.

The allowance for development length and shear in multi-column overhangs not being considered is from research Report 52-1F [Ferguson, 1964]. When the exterior beam is placed between 0.5d to 1.2d from the effective face of the column on the cap overhang, the research showed that the load was directly strutting to the column.

Longitudinal reinforcement for shear check can generally be ignored for typical multicolumn bents based on an interpretation of <Article 5.7.3.5> along with research Report 52-1F [Ferguson, 1964] for multi-column bents.

Where there is a beam on a cap overhang near a column, the beam creates a clamping force on the cap. If there is no beam on the overhang or if the exterior beam is closer than 0.5d from the exterior column, the beam no longer provides a clamping effect, so the longitudinal reinforcing checks in <Article 5.7.3.5> should be verified.

<Article 5.7.3.6.3> does not usually apply to standard (rectangular) multi-column bent caps due to the little amount of torsion in them. Inverted-tee bents may see a significant amount of torsion requiring <Article 5.7.3.6.3> to be verified. Whether or not to perform the longitudinal reinforcing check should be based on engineering judgement.

Lap lengths for bundled bars or bars spaced closely in either direction may need a longer development length than shown in the TxDOT *Bridge Detailing Guide*; see <Article 5.10.8.2.3> for modifications for bundled bars, <Article 5.10.8.4.2a> for how to treat the lap of individual bars in a bundle, and <Article 5.10.8.2.1c> for the calculation of λ_{rc}.

Bars with yield strengths above 60 ksi will need lap lengths longer than shown in the TxDOT *Bridge Detailing Guide*; see <Article 5.10.8.2.1> for the tension development length and <Article 5.10.8.4> for lap length requirements.

References:

Ferguson, Phil M., "Design Criteria for Overhanging Ends of Bent Caps-Bond and Shear," Report No. 52-1F, Center for Transportation Research, The University of Texas at Austin, Austin, TX., (https://library.ctr.utexas.edu/digitized/texasarchive/phase1/52-1f-ctr.pdf), 1964.

6.6.1.5 Detailing

Provide maximum spacing of 12 inches for flexural reinforcement.

Provide vertical clear space between bars of at least 4 inches.

Provide horizontal clear space between bars of at least 3 inches.

Provide horizontal tie bars to the vertical stirrup legs to support each additional row of flexural reinforcement.

Use #5 or #6 stirrups with a 4-inch minimum and 12-inch maximum spacing. If required spacing is less than 4 inches, use double stirrups.

For caps required to be investigated for torsion according to <Article 5.7.2.1>, detail stirrup reinforcing according to <Article 5.10.8.2.6d> including providing the 135-degree standard hooks.

For flexural reinforcement, use #11 bars. For trestle bents or to satisfy development requirements, small bars may be used. Do not mix bar sizes.

In heavily reinforced caps, bundling bars in two-bar bundles may be used to maintain necessary clear spacing. Adjust lap lengths, development lengths, and clear spacing as required by AASTHO.

Use longitudinal skin reinforcement in accordance with <Equation 5.6.7-3> in caps deeper than 3 feet. Bent caps that are 3 feet or less in height, provide a minimum of two #5 bars equally spaced in each side face.

Lap reinforcement bars longer than 60 feet. Locate these laps in compression or low-tension zones. Stagger or alternate laps in adjacent reinforcement bars to minimize congestion. Mechanical couplers or welded splices may be used for phased construction.

Detail bearing seats on the bent details. Resolve any geometric conflicts with the bent ends.

Use dowels at each bent for single span I-Girders (TxGirders) and spread box beam (X-Beam) bridges.

Do not use dowels at the ends of multi-span TxGirders and X-Beams units.

Use at least 2 dowels per fixed span end. Place dowels at the outside TxGirders and X-Beams. When the distance between the centerlines of the outside girders exceeds 80 feet, place the dowels at an inside girder. Keep the distance between the outermost dowels at a maximum of 80 feet (+/-) apart.

Commentary

The minimum spacing for reinforcing allows for adequate consolidation of concrete.

Extreme grades and skews can produce conflicts between beam ends and bearing build-up of adjacent spans, as well as beam ends and bearing build-ups with transition walls or pedestals on bent caps.

The TxDOT *Bridge Detailing Guide* shows typical bearing seat configurations.

6.6.1.6 Precast Bent Caps

Individual precast elements should be sized to weigh less than the below transport and lifting weight limits:

- ≤ 100 tons for transport over distance.
- ≤ 150 tons for erection assuming a two-crane lift.

Evaluate cross-sections and feasible lift points to prevent cracking. Reasonable assumptions for impact loads and form suction loading should be considered.

Precast bent caps may be conventionally reinforced, pretensioned, and/or post-tensioned.

When using closure pours, design for and select materials that limit the shrinkage restraint cracking due to casting against existing concrete.

6.6.2 Inverted-Tee Reinforced Concrete Bent Caps

6.6.2.1 Materials

Use normal weight concrete Class F, H, or K concrete (f_c' = 5.0 ksi). Higher concrete strengths may be required in special cases. Mass concrete may be needed depending on the size of the cap.

Use Grade 60 reinforcing steel. Higher reinforcing steel grades may be used provided their use satisfies requirements in AASHTO.

6.6.2.2 Geometric Constraints

Minimize the variations in inverted-tee cross section dimensions within a project.

Keep the top of stem below the bottom of the slab as shown on the Miscellaneous Slab Standard sheet for the girder type used; see standard drawings IGMS (TxGirders) and SGMS (steel girders).

Use a minimum ledge width that provides a minimum of 3 inches from the edge of the bearing pad to the edge of the cap. On skewed bents, pad orientation must be accounted for and the ledge widened accordingly.

Provide a stem width with a minimum of 3 inches wider than the supporting columns to allow column reinforcing to extend into the cap without bending.

Use a stem height in whole inch increments.

6.6.2.3 Structural Analysis

In lieu of a more detailed analysis, it is permissible to analyze multi-column caps as simply supported beams on knife-edge supports at the center of piling or columns. If the column is wider than 4 feet, use a model that takes the stiffness of the column into consideration.

See Section 6.6.1.3 for dead load and live load modelling.

Design for torsion in bents due to the bearings being relatively far from the center of the cap; differences in adjacent span lengths or beam spacing; and when construction phasing may result in one side of the cap be loaded.

Show limitations to the construction sequence on the plans. In some instances, the unbalanced dead load during construction could cause the controlling torsional force in the cap and on the cap to column connection.

6.6.2.4 Design Criteria

Check limit states using Strength I.

Check 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 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 $0.6f_y$.

When calculating spacing requirements per <Article 5.10.3.1> use the absolute maximum aggregate size, the smallest sieve with 100% passing, not the maximum nominal size of the aggregate per Item 421, "Hydraulic Cement Concrete". If smaller reinforcing spacing is required, limit the sizes of aggregate in the material notes on the plan sheet. Contact Bridge Division for guidance.

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-foot wide or wider, take design negative moments at the effective face of the column.

For typical multi-column bent caps supporting multiple beams, strut-and-tie modeling provisions of <Article 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 method.

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>.

Size web reinforcing for hanger loads, vertical shear, and torsion.

Minimize the number of stirrup spacing changes.

Using <Appendix B5> to design shear for conventionally reinforced caps is the preferred method. <Article 5.7.3.4.2> may be used for prestressed caps.

If <Article 5.7.3.4.1> or <Article 5.7.3.4.2> is used, ensure that the minimum stirrup spacing requirements are not violated. If the required stirrup spacing is less than the minimum allowed per AASHTO, re-evaluate using <Appendix B5>, before increasing the shear depth.

Base lap lengths on tension lap requirements <Article 5.10.8.4.3>.

Design and reinforce bearing seat build-ups taller than 3 inches.

Limit f_y to 60 ksi for ledge and hanger resistance calculations in <Article 5.8.4.3> and <Article 5.8.4.2>.

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 and $(W + 3a_v)$ shall not exceed S for interior bearing and $S \leq 2c$ for exterior bearings.

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

Verify the ledge width provides sufficient development length for the ledge reinforcing as shown in Figure 6.5.2.4-1.

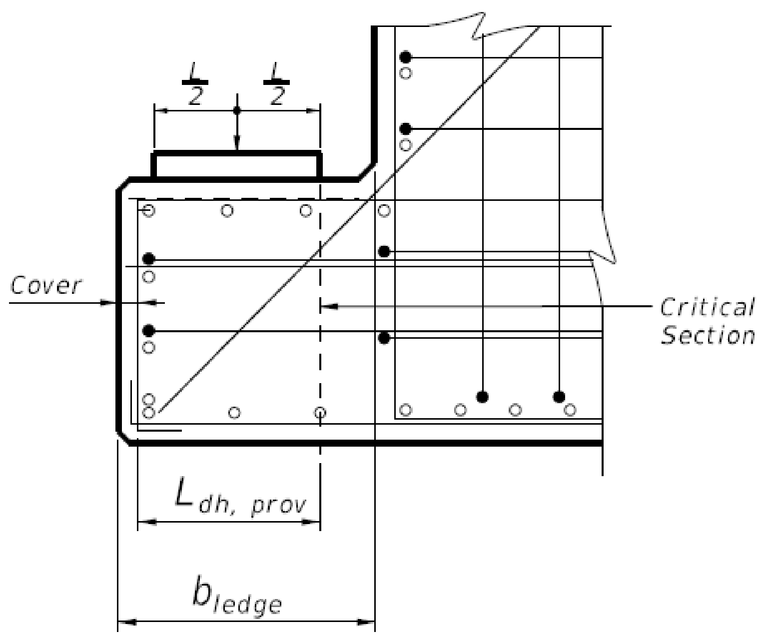


Figure 6.6.2.4-1: Ledge Width

Extend caps at least 2.5 feet past centerline of the exterior beam to prevent excessive hanger and ledge reinforcing requirements and to provide adequate punching shear capacity. For skewed bridges or phased designs, consider punching shear capacity for the exterior beams; a cap extension of 2.5 feet may not be adequate.

Commentary

Provide a design where all the tensile reinforcement yields at ultimate, which results in a ductile failure. For example, when there are 3 or more mats of reinforcement on the top or bottom of the bent cap, the reinforcement in the third or more mats will not yield. Bridge Division recommends using the strain compatibility design method to get the capacity of the bent cap.

Simplified procedure in <Article 5.7.3.4.1> for all designs and the general procedure in <Article 5.7.3.4.2> for nonprestressed caps result in overly conservative shear designs.

Longitudinal reinforcing check, <Article 5.7.3.5>, must pass for all inverted-tees. Beam loads on an inverted-tee are applied at the bottom of the section; they do not provide the clamping effect that is exhibited with beam loads applied to the top of a rectangular cap.

Lap lengths for bundled bars or bars spaced closely in either direction may need a longer development length than shown in the TxDOT Bridge Detailing Guide; see <Article 5.10.8.2.3> for modifications for bundled bars, <Article 5.10.8.4.2a> for how to treat the lap of individual bars in a bundle, and <Article 5.10.8.2.1c> for the calculation of reinforcement confinement factor (λ_{rc}).

Bars with yield strengths above 60 ksi will need lap lengths longer than shown in the TxDOT Bridge Detailing Guide; see <Article 5.10.8.2.1> for the tension development length and <Article 5.10.8.4> for lap length requirements.

6.6.2.5 Detailing

Provide a maximum spacing of 12 inches for flexural reinforcement.

Provide a vertical clear space between bars of at least 4 inches.

Provide a horizontal clear space between bars of at least 3 inches.

Provide horizontal tie bars to the vertical stirrup legs to support each additional row of flexural reinforcement.

Provide extra vertical and horizontal reinforcing across the end surfaces of the stem to resist cracking. Single #5 bars, anchored at each end with hooks, 6-inch (+/−) spacing for vertical bars, and horizontal bars spaced with horizontal temperature and shrinkage bars 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.

Provide diagonal #7 bars at 6-inch spacing for 2.5 feet at the end of ledge. Place the reinforcing such that it is developed at the point it intersects the crack at the ledge-stem interface. See Figure 6.5.2.5-1 for end of cap reinforcing and the TxDOT Bridge Detailing Guide for additional information.

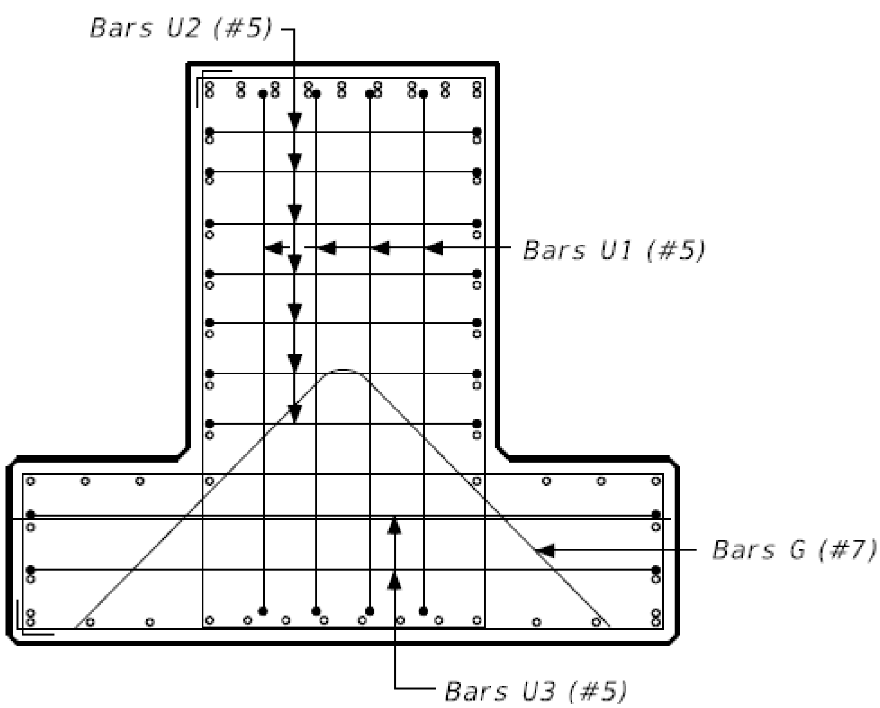


Figure 6.6.2.5-1: End of Cap Reinforcing

Use stirrups with a 4-inch minimum and 12-inch maximum spacing. If required spacing is less than 4 inches, use double stirrups.

If torsional resistance is explicitly addressed in the design, ensure the stirrup detailing is consistent with AASHTO requirements.

Use longitudinal skin reinforcement in accordance with <Equation 5.6.7-3> in caps deeper than 3 feet.

Lap reinforcement bars longer than 60 feet. Locate these laps in compression or low-tension zones. Stagger or alternate laps in adjacent reinforcement bars to minimize congestion. Mechanical couplers or welded splices may be used for phased construction.

Detail bearing seats on the bent details. Resolve any geometric conflicts with the bent ends.

Space stirrups (Bars S and SS) and ledge reinforcing (Bars M and N) such that the spacing is multiples of the other to allow for grouping of the bars.

Cap hanger and ledge reinforcement may be aligned to match the cap skew when simpler detailing can be achieved as shown in Figure 6.6.4.5-2. For larger skewed caps, this detail is preferred rather than transitioning normally oriented hanger and ledge reinforcement by fanning the reinforcement at the cap ends when a skewed end detail is needed as shown in Figure 6.6.4.5-2.

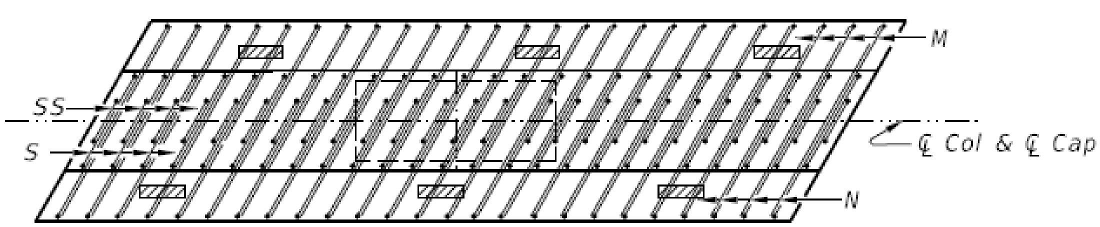


Figure 6.6.2.5-2: Skewed Reinforcing

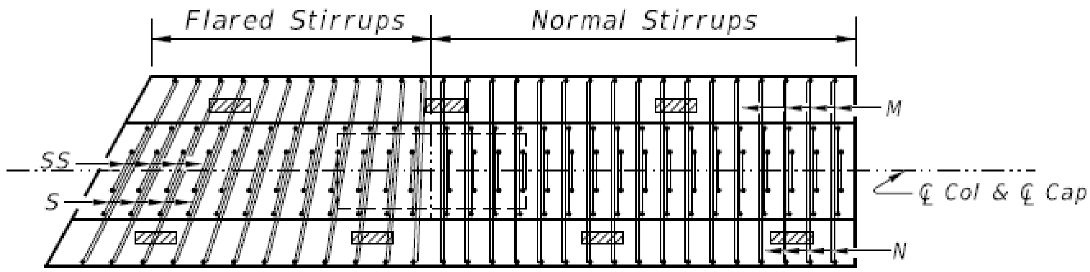


Figure 6.6.2.5-3: Flared Reinforcing

For caps required to be investigated for torsion according to <Article 5.7.2.1>, detail stirrup reinforcing according to <Article 5.10.8.2.6d> including providing the 135-degree standard hooks.

Use dowels at each bent for single span I-Girders (TxGirders) and spread box beam (X-Beam) bridges.

Do not use dowels at the ends of multi-span TxGirders and X-Beams units.

Use at least 2 dowels per fixed span end. Place dowels at the outside TxGirders and X-Beams. When the distance between the centerlines of the outside girders exceeds 80 feet, place the dowels at an inside girder. Keep the distance between the outermost dowels at a maximum of 80 feet (+/-) apart.

For inverted-tee bent caps with two expansion joints, slab dowels are used to provide continuity between the inverted-tee cap and the slab. See the Miscellaneous Slab Details for I-Girders (IGMS) and steel girders (SGMS) standard details for more information.

Commentary

The minimum spacing for reinforcing allows for adequate consolidation of concrete.

Extreme grades and skews can produce conflicts between beam ends and bearing build-ups the stem.

6.6.2.6 Precast Inverted-Tee Bent Caps

Individual precast elements should be sized to weigh less than the below transport and lifting weight limits:

- ≤ 100 tons for transport over distance.
- ≤ 150 tons for erection assuming a two-crane lift.

Evaluate cross-sections and feasible lift points to prevent cracking. Reasonable assumptions for impact loads and form suction loading should be considered.

Precast inverted-tee bent caps may be conventionally reinforced, pretensioned, and/or post-tensioned.

6.6.3 Post-Tensioned Concrete Bent Caps

6.6.3.1 Materials

Use normal weight concrete Class F, H, or K concrete (f_c' = 5.0 ksi). Higher concrete strengths may be required in special cases. Mass concrete may be needed depending on the size of the cap.

Use Grade 60 reinforcing steel. Higher reinforcing steel grades may be used provided their use satisfies requirements in AASHTO.

Use 0.6-inch 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*.

6.6.3.2 Geometric Constraints

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

Use a cap depth greater than the cap width, except as needed for the situations listed below:

- When widening a cap to meet the minimum support length per <Article 4.7.4.4>.
- When widening a cap to accommodate a cap to column connection when one or both elements are precast.
- Meeting the vertical clearance needs of a lower roadway.

Cap width should be a minimum of 3 inches wider than the supporting columns to allow column reinforcing to extend into the cap without bending.

See Section 6.6.1.2 for minimum cap widths based on beam or girder type for rectangular caps. See Section 6.6.2.2 for inverted-tees.

6.6.3.3 Structural Analysis

Detail column-to-cap connections to reflect assumptions of fixity made for post-tensioned cap design.

Account for column stiffness and secondary force effects due to post-tensioning.

See Section 6.5.1.3 for dead load and live load modelling.

6.6.3.4 Design Criteria

Limit tendon stress before anchor set to no more than the lesser of $0.77f_{\mathrm{pu}}$ and the stress limits in <Article 5.9.2.2> for low-relaxation strands.

Determine prestress losses from elastic shortening, creep, shrinkage, and relaxation as prescribed for prestressed concrete in Section 5.3.1.

Use a minimum of 4 tendons in a cap.

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

Check limit states using the Strength I load combination.

Check the Service I load combination for both tension and compression stress.

Minimize the number of stirrup spacing changes.

For typical multi-column bent caps supporting multiple beams, strut-and-tie modeling provisions of <Articles 5.8.2> 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 method.

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

See Section 6.6.1.4 for shear requirements. See Section 6.6.2.4 for shear ledge and hanger requirements.

6.6.3.5 Detailing

Provide a minimum 5-foot tangent length of tendon from anchorage bearing plate to initial point of curvature. Do not exceed the minimum radius of curvature for ducts based on published values from approved vendors.

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 of critical section(s) (i.e. the location of maximum flexural demand). Provide the magnitude of the initial stress in each tendon after anchor set at the critical section(s). Provide the assumed post-tensioning stress and long-term loss of each tendon at the critical section(s).

Provide the stressing and erection sequence on the plans, including form removal and girder placement. Define when the bottom cap forms can be removed, based on the construction sequence designed.

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 = $0.XX\ x\ f_{pu}\ x\ A_{ps} = yy\ kips$; replace "XX", "$f_{pu}$", "$A_{ps}$", and "$yy$" with the values used in design.
- Include stressing sequence, including constraints on partial stressing.

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.

See Section 6.6.1.5 (rectangular bent caps) and Section 6.6.2.5 (inverted-tee bent caps) for detailing requirements. For flexural reinforcement, use #9 or larger bars.

6.6.4 Steel Straddle Bent Caps

6.6.4.1 Overview

The use of steel straddle bent caps requires Bridge Division approval.

Design steel straddle bent caps for redundancy. Without explicit consideration of redundancy in the design process, this type of bridge component is classified as Nonredundant Steel Tension Member (NSTM).

Steel bent caps consisting of more than 2 columns are considered redundant.

Triple steel I-girder straddle bent caps are considered redundant.

Refer to the TxDOT *Preferred Practices for Steel Bridge Design, Fabrication, and Inspection* for more guidance on steel bent caps.

Commentary

Steel straddle bent caps, consisting of 2 columns, are often utilized in congested urban environments when intersecting roads do not permit the use of conventional piers. These caps can be used as an accelerated bridge construction technique to minimize impact to motorists, manage vertical clearance limitations, and/or limit weight of prefabricated elements.

The National Bridge Inspection Standards (NBIS) require hands-on inspection of NSTMs of in-service bridges (23 CFR 650.313(f)(2)) every two years and must be fabricated to the provisions of Clause 12 specified in AWS D1.5 Bridge Welding Code.

Background on redundancy requirements can be found in FHWA Memorandum_ACTION: Inspection of Nonredundant Steel Tension Members (dot.gov).

Internal redundancy is a redundancy that exists within a primary member cross-section without load path redundancy such that fracture of one component will not propagate through the entire member, is discoverable by the applicable inspection procedures, and will not cause a portion of or the entire bridge to collapse (United States Code of Federal Regulations, Title 23 Highways, Part 650.305 (23 CFR 650.305)). Typically, methods to establish internal redundancy demonstrate that the remaining structural section of the member will continue to safely carry dead and live load until the fracture is discovered, presumably at the next inspection event. Previous research efforts have demonstrated the ability of sections built up from mechanically fastened components in certain structure types to prevent fracture of one element from propagating to connected elements and to redistribute stress within the section.

TxDOT research project 0-7012, Development of Non-Fracture Critical Steel Box Straddle Caps gives guidance on designing and detailing steel straddle caps for redundancy.

To help demonstrate redundancy, consider designing steel bent caps with a bolted bottom flange, which is a mechanically fastened component.

References:

Zecchin, E., E. Williams, C. Liang, T. Helwig, M. Engelhardt, E. Williamson, and M. Hebdon, "Design of Internally Redundant Steel Box Straddle Caps," Report No. FHWA/TX-24/0-7012-1, Center for Transportation Research, The University of Texas at Austin, Austin, TX., (https://library.ctr.utexas.edu/ctr-publications/0-7012-1.pdf), 2023.

6.6.4.2 Design Criteria

All steel straddle bent caps must satisfy the requirements in the manual and must be evaluated for redundancy according to AASHTO Guide Specifications for Internal Redundancy of Mechanically-Fastened Built-up Steel Members or AASHTO Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members, as applicable.

Members or portions that would otherwise be classified as Nonredundant Steel Tension Members (NSTM) when evaluated based on load path redundancy alone, shall instead be designated in the contract documents as SRMs (system redundant members) or IRMs (Internal redundant members) and need not be subject to the hands-on in-service inspection protocol for NSTM as described in 23 CFR 650. The SRMs or IRMs shall be fabricated according to the American Welding Society (AWS) D1.5 Bridge Welding Code - Fracture Control Plan (FCP).

6.6.4.3 Submittal and Approval

To satisfy FHWA requirements, TxDOT Bridge Division must approve each steel straddle bent cap design for internal redundancy and system redundancy. At 60% plan submittal, 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 straddle bent cap plan sheets.
- Steel straddle bent cap design calculation package meeting the requirements of <Chapter 6>. The package shall include the calculations demonstrating redundancy and an explanation of assumptions for modeling and modeling method used if a refined method is chosen.

Upon TxDOT's acceptance and approval of 100% plans, submit final calculation package in accordance with Chapter 2, including final redundancy calculations, as well as the full completed refined analysis records/computer models as this information will be retained and included with the bridge inspection management system.

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

6.7 Abutments

This section documents policy on conventional abutments and semi-integral abutments.

6.7.1 Abutments

6.7.1.1 Materials

Use normal weight concrete Class C concrete (f_c' = 3.6 ksi). Higher concrete strengths may be required in special cases. If higher concrete strengths are required, use Class F, H, or K.

Use Grade 60 reinforcing steel. Higher reinforcing steel grades may be used provided their use satisfies requirements in AASHTO.

6.7.1.2 Geometric Constraints

Position the backwall, wing wall lengths, wing wall support, and various other standardized items as shown in the TxDOT *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.

For abutments supporting slab beams and simply supported steel beams, use a width of at least 2.75 feet and 30-inch diameter drilled shafts as a minimum.

For abutments supporting Tx28 through Tx54 girders, adjacent box beams, and X-beams use a width of at least 3.50 feet and 36-inch diameter drilled shafts as a minimum.

For abutments supporting Tx62 and Tx70 girders, use a cap width of at least 4.00 feet and 42-inch diameter drilled shafts as a minimum.

Maximum spacing of abutment drilled shafts or pile groups:

- TxGirders less than or equal to 40 inches in depth, use 13.50 feet.
- TxGirders greater than 40 inches in depth, use 11.00 feet.
- Steel girders greater than 70 inches in depth, use 11.00 feet, unless lesser spacing is required by analysis.
- All other beam types less than or equal to 40 inches in depth, use 16.00 feet.
- All other beam types greater than 40 inches in depth, use 12.50 feet.

6.7.1.3 Structural Analysis

Analysis of abutments is similar to rectangular bent caps. Refer to Section 6.6.1 for information on the structural analysis and design criteria.

Structural analysis is generally not required for abutments within the geometric constraints, such as cap width and height; backwall heights and thicknesses; wingwall lengths; and drilled shaft spacing, shown on the bridge standards. Structural analysis is required for abutments that are not similar to those shown on the bridge standards.

Structural analysis is required where abutments have significant likelihood of acting as a bent due to scour and stream migration or there are known future expansion plans that would result in lengthening the bridge.

Drilled shaft loads may be calculated as the total vertical load on the cap divided equally among the cap shafts.

Wingwall shaft or pile load is usually taken as 10 to 15 tons per shaft or pile, unless calculated vertical loads are higher. Wingwall loads should account for the portion of the wingwall and bridge rail it is supporting at a minimum.

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 and details shown on standard drawings.

Vertical piles may be used if adequate restraint to lateral loads can be achieved. Examples of adequate restraint are slab spans and pan form spans that are doweled into the abutment.

Vertical piles may be used in Mechanically Stabilized Earth (MSE) wall backfill, when analysis determines adequate resistance to lateral loads can be achieved. Do not batter 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.

Calculate pile loads as the total vertical load on the cap divided equally among the cap pilings.

For stub abutments (typical TxDOT abutment type), 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 $\Delta p = k y_s h_{eq}$, where k = 0.25, y_s = 120 pcf. For abutments with d < 5 feet, take h_{eq} = 4.0 feet. For all other abutment types, see <Table 3.11.6.4-1>. Retaining type abutments in questionable soils may justify a more rigorous analysis.

For abutments with battered piling, design for the combined horizontal force and vertical load. Do not allow the back pile to go into tension due to the lateral load. When checking for tension, design for dead load and soil pressure only.

6.7.1.4 Design Criteria

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 inches.

6.7.1.5 Detailing

When retaining walls run parallel to the roadway, the abutment cap and wingwalls are impacted by the wall panels and coping. Refer to the TxDOT *Bridge Detailing Guide* for how this is typically shown.

Use dowels at abutments for single span I-Girders (TxGirders) and spread box beam (X-Beam) bridges.

Do not use dowels at the ends of multi-span TxGirders and X-Beams units.

Use at least 2 dowels per fixed span end. Place dowels at the outside TxGirders and X-Beams. When the distance between the centerlines of the outside girders exceeds 80 feet, place the dowels at an inside girder. Keep the distance between the outermost dowels at a maximum of 80 feet (+/-) apart.

6.7.1.6 Precast Abutment Caps

Individual precast elements should be sized to weigh less than the below transport and lifting weight limits:

- ≤ 100 tons for transport over distance.
- ≤ 150 tons for erection assuming a two-crane lift.

Evaluate cross-sections and feasible lift points to prevent cracking. Reasonable assumptions for impact loads and form suction loading should be considered.

Precast abutment caps may be conventionally reinforced, pretensioned, and/or post-tensioned.

Account for the backwall location in developing the connection details. Backwalls may be moved relative to the typical location of the backwall face along the centerline of the abutment cap.

Design for the stability of the abutment prior to establishing connections. The backwall can cause the abutment to rotate.

Account for the backfill type and construction loading due to placing and compacting the backfill.

Attaching the wingwall to the precast abutment has proven to be difficult in the field. Bolting has not been fully successful. One option is to construct cast-in-place wingwalls off the critical path. A design option is to extend the abutment longitudinally to capture the transverse approach cross section (usually limited to shallower superstructures).

6.7.2 Semi-Integral Abutments

Contact Bridge Division if semi-integral abutments are being considered to discuss the circumstance and application. Semi-integral abutments are reliant on the backfill material and configurations.

Semi-integral abutments cannot be used in conjunction with retaining walls.

See Section 6.7.1 for other design criteria.

For more information see TxDOT Research Project 0-6936 Development of Integral/Semi-Integral Abutments for Texas Bridges.

Commentary

References:

Zornberg, J.; B. Mofarraj, T. Helwig, and J. Walter, "Development of Integral/Semi-Integral Abutments for Texas Bridges," Report No. FHWA/TX-24/0-6936-1, Center for Transportation Research, The University of Texas at Austin, Austin, TX., (https://library.ctr.utexas.edu/ctr-publications/0-6936-1.pdf), 2023.

Chapter 7 Concrete Culverts

7.1 Overview

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

7.2 Materials

For cast-in-place concrete culverts, use normal weight Class C concrete (f_c' = 3.6 ksi) with the following exceptions: use normal weight 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).

For precast concrete culverts, use normal weight 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 thus required, use high performance concrete (HPC) concrete and/or galvanized reinforcement (refer also to Item 440, “Reinforcement for Concrete”).

7.3 Geometric Constraints

The maximum skew angle for box culverts is 45°.

Provide 1.5 inches of clear cover for reinforcement in cast-in-place concrete culverts.

Provide 1.0 inch of clear cover for reinforcement in precast concrete culverts.

Provide 2.0 inches of clear cover above the top layer of reinforcement in the top slab of direct traffic concrete culverts.

7.4 Structural Analysis

Analyze each box culvert as a simply supported two-dimensional frame pinned at one support only and free to move horizontally at all other supports. 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 feet. Otherwise, distribute the curb and railing loads over 16 feet.

7.5 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 feet. Only consider those axle loads which are applied directly above the culvert. If the axle load acts beyond the middle of an exterior wall, ignore that entire axle load.

Unless site-specific information is available, assume cohesionless soil with unit weight equal to 120 pcf and friction angle equal to 30°. Use an at-rest lateral earth pressure coefficient, $k_{o,}$ of 0.33 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.

7.6 Detailing

Place reinforcement for skewed ends as directed in the Single Box Culvert Miscellaneous Details (SCC-MD and SCP-MD) or Multiple Box Culvert Miscellaneous Details (MC-MD) standard sheets.

Chapter 8 Ancillary Structures

8.1 Overview

This chapter applies to traffic structures under the purview of Bridge Division: sign structures, high mast and roadway illumination poles, traffic signal poles, and intelligent transportation services poles.

Design all non-standard ancillary structures *to AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 6th Edition (LTS-6) with 2020 interims* or *AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals 1st Edition 2015 (LRFDLTS-1) with 2020 interims.*

8.2 Materials

Use normal weight concrete Class C concrete (f_c' = 3.6 ksi).

Use Grade 60 reinforcing steel.

For overhead and cantilever sign support truss members, use 'Carbon Steel' or 'Low-alloy Steel' material as defined in Item 442, Metal for Structures, unless otherwise specified in project plans.

For anchor bolt templates, use ASTM A36. Galvanizing is not required for anchor bolt templates. Templates are typically used to help align the anchor bolts and are cast-in-concrete. These are considered nonstructural.

Use connection bolts meeting ASTM F3125 Grade A325 or A490 unless otherwise specified. Stainless steel may be used for the J bolt for the Digital message sign (DMS) mounts.

Table 8.2-1 provides material per element and structure type when not consistent across structure types. All references are to ASTM standard materials.

Table 8.2-1: Variable Material per Traffic Structures

Item	Overhead Sign Structures and Monotubes	High Mast Illumination Poles	Intelligent System Transportation Poles	Traffic Signal Poles	Roadway Illumination Poles
Pole Shaft	A53 Gr. B A500 Gr. B A501	A709 Gr. 50 A572 Gr. 50	A572 Gr. 50	A572 Gr. 50 A595 Gr. A A588 A1011/A1008 HSLAS Gr. 50 Cl. 2	A572 Gr. 50 A595 Gr. A A1011/A1008 HSLAS Gr. 50 Cl. 2
Baseplate or Handhole	A36 A588 A572 Gr. 50	A709 Gr. 50 A572 Gr. 50 A633 Gr. C	A36 A572 Gr. 50	A36 A572 Gr. 50	A36 A572 Gr. 50
Anchor Bolts	F1554 Gr. 55, Gr. 105 A193 Gr. B7	F1554 Gr. 55, Gr. 105 A193 Gr. B7	F1554 Gr. 55 A193 B7	F1554 Gr. 55 *For D < 1″*, A36	F1554 Gr. 55 A193 Gr. B7

As per Item 440, tubing that meets American Petroleum Institute (API) Standards may be used if produced by a mill listed in the standard API specifications as authorized to produce pipe with the API monogram. Commonly used API materials are 5L Grade X52 as a substitute to ASTM 500 Gr B, API Standard 5L Grade X42 and ASTM A252 Grade 2 Type E or S material for monotubes, and ASTM A252 Grade 3 for cantilever overhead sign structures (COSS) columns.

8.3 Geometric Constraints

Design overhead sign structures to meet or exceed the vertical clearances required in the *TxDOT Roadway Design Manual*.

Material availability restricts thickness of monotube pipe to 0.75 inches maximum.

8.4 Structural Analysis

Use category I for fatigue analysis.

8.5 Design Criteria

Alternate design criteria for Roadway Illumination Poles (RIP) can be found in the RIP standard sheets.

Alternate design criteria for Cantilever Overhead Sign Structures (COSS) can be found in Item 650 “Overhead Sign Supports”.

8.6 Detailing

For all overhead sign structures, provide a corresponding elevation view that details: total sign area, maximum allowable sign area (for future sign changes), applicable standard being used, foundation diameter and lengths, and structure station. For nonstandard sign structures provide details for all structural members and connections.

All structures located in clear zones, such as luminaires (roadway illumination poles) and roadside signs, are required to be protected by guardrails or have breakaway supports, if applicable. Place those without breakaway supports behind a guardrail. See the *TxDOT Roadway Design Manual* for more information on clear zones.

Do not grout the base of ancillary structures. Grouting can lead to water entrapment and decreased life of anchor bolts.

8.7 Appropriate Use of Standards

8.7.1 Overhead Sign Structures

Refer to Sign Mounting Details (SMD(2) series) Overhead Signs standards for sign attachment details including vertical placement of sign and sign bracket spacing. If using sign placement and brackets different from the standard, design of connection and brackets will be required.

Cantilever Overhead Sign Support and Overhead Sign Bridge (COSS/OSB) standards have different standards for wind heights up to 30 feet and standards for wind heights between 30 feet and 50 feet. Use the appropriate standards based on the average ground elevation near the structure.

Use the standards applicable to the wind loading of the region in which the structure is located. If a structure is located in a region adjacent to another with higher wind speeds, consider using the higher wind zone. Some standards may not cover the wind zone for the structure. For example, the monotube standards cover 90 mph and 130 mph wind speeds.

8.7.2 Traffic Signal Pole Structures

Acceptable attachment arrangements for the Single Mast Arm (SMA) and Long Mast Arm (LMA) Assembly standards can be determined by comparing the first moment of area assumed in the standard design to that of the proposed configuration.

If the first moment of area for the proposed attachment configuration (Q_p) is less than or equal to the standard first moment of area (Q_s), then the proposed configuration may be used on the standard structure. If Q_p is greater than Q_s, contact Bridge Division for a more refined analysis.

The standard first moment of area (Q_s) can be determined using Table 8.7.2-1 and Table 8.7.2-2.

Table 8.7.2-1: Standard First Moment of Area (Q_s)

Mast Arm Length (feet)	Q_s* (cf)
20	648
24	778
28	907
32	1,037
36	1,166
40	1,296
44	1,426
48	1,555

*Assumes an Effective Projected Area = 32.4 sq ft

Table 8.7.2-2: LMA Standard First Moment of Area (Q_s)

Mast Arm Length (feet)	Q_s* (cf)
50	2,600
55	2,860
60	3,120
65	3,380

*Assumes an Effective Projected Area = 52.0 sq ft

The first moment of area (Q_p) for the proposed attachment configuration (see Figure 8.7.2-1) is determined by adding together all of the effective projected areas (EPA) of the individual attachments multiplied by the distance of their respective centroids from the pole centerline (x):

$$Q_p = \sum \{(x_i) \cdot (EPA_i)\}$$

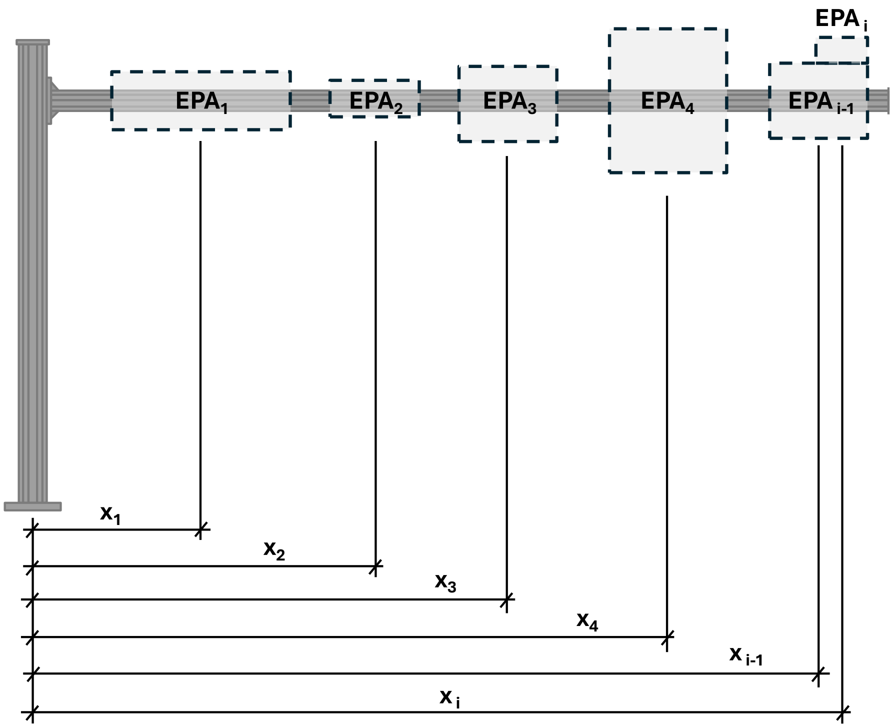


Figure 8.7.2-1: Variables for First Moment of Area

The effective projected area (EPA) is determined by multiplying the actual attachment area (length x width) by an assigned drag coefficient (C_d).

$$EPA_i = (C_d) \cdot (Actual\ Attachment\ Area)$$

The drag coefficients are based on attachment type and are found in Table 8.7.2-3.

Table 8.7.2-3: Drag Coefficients (C_d) for Various TSS Attachments

Attachment	C_d
Traffic Signal	1.20
Flashing Beacon	1.20
Square Sign (Length/Width = 1)	1.12
All Other Signs	1.20

Appendix A TxGirder Haunch Design Guide

A.1 Components of Haunch

A.1.1 Camber

Camber is the upward deflection in the girder after release of the prestressing strands due to the eccentricity of the force in the strands. The camber of the girder is usually the largest contribution to haunch. In most cases, as camber increases, so does haunch.

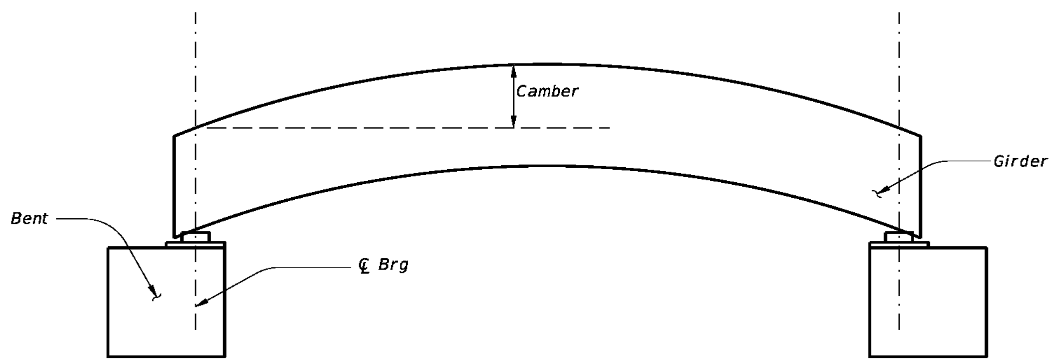


Figure A.1.1-1: Camber of Girder (Before Slab is Placed)

A.1.2 Dead Load Deflection

The dead load deflection used in haunch calculations is the deflection due to the dead load of the slab only (it does not account for haunch weight). The dead load helps lessen the haunch. As the dead load deflection increases, the haunch decreases. Note: The dead load deflection calculation assumes a monolithic, cast-in-place slab, whether or not prestressed concrete panels are used. Where prestressed concrete panels are used deflections used to screed the roadway surface will be different since the deflection due to the panels will already be there.

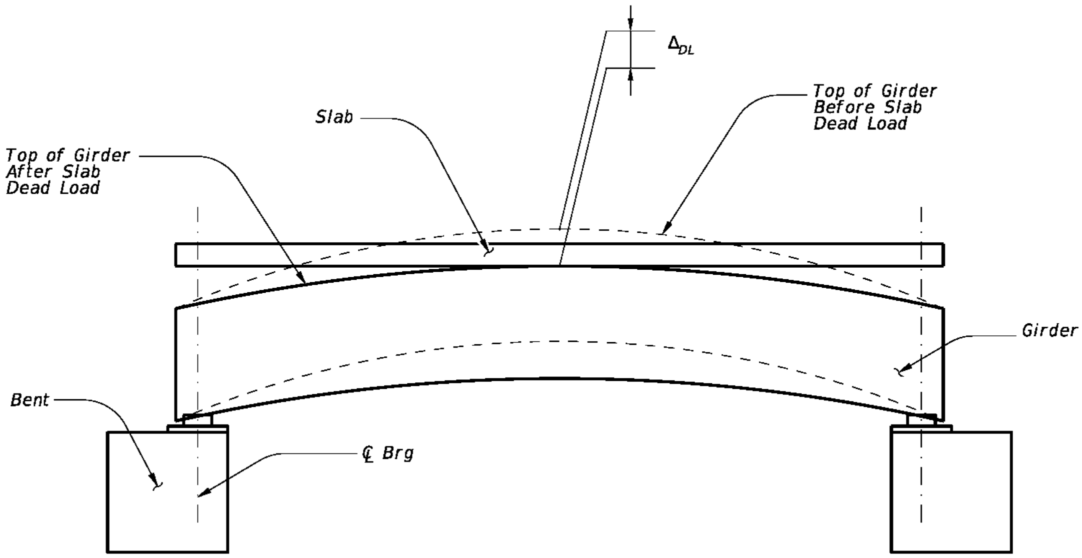


Figure A.1.2-1: Girder After Slab is Placed

A.1.3 Cross Slope

The cross slope is the slope of the slab across the transverse section of the girder. The cross slope correction (CSC) is the distance from the bottom of the slab to the top of the girder at the center of the top flange needed to prevent encroachment of the girder into the slab at the lowest point of the cross slope. The CSC is needed for TxGirders since the girders are placed vertically on the bearing pads. As CSC increases, the haunch also increases.

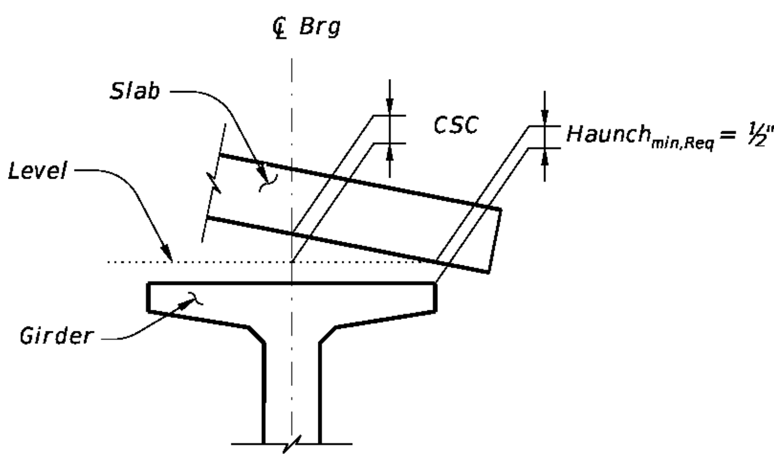


Figure A.1.3-1: Cross Slope Correction

A.1.4 Vertical Clearance Ordinate (VCO)

The VCO is the distance from the Bridge Geometry System (BGS) reference line to the roadway surface. The reference line is the chorded roadway surface between the center of bearings. The VCO is given in the BGS output as negative when the roadway surface is above the reference line and positive when the roadway surface is below the reference line. When the VCO is negative (crest curve) the haunch decreases at center of bearing, and when the VCO is positive (sag curve) the haunch increases at center of bearing.

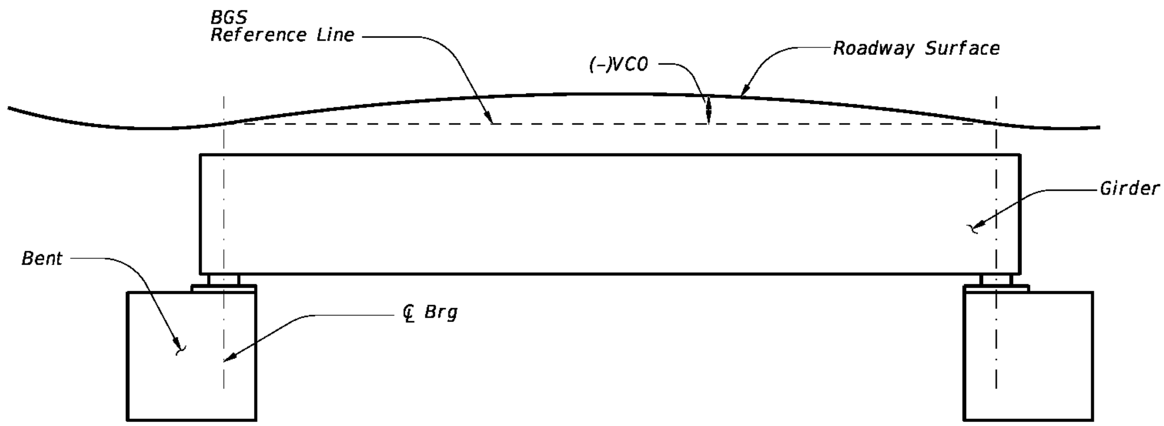


Figure A.1.4-1: VCO with Respect to BGS Reference Line

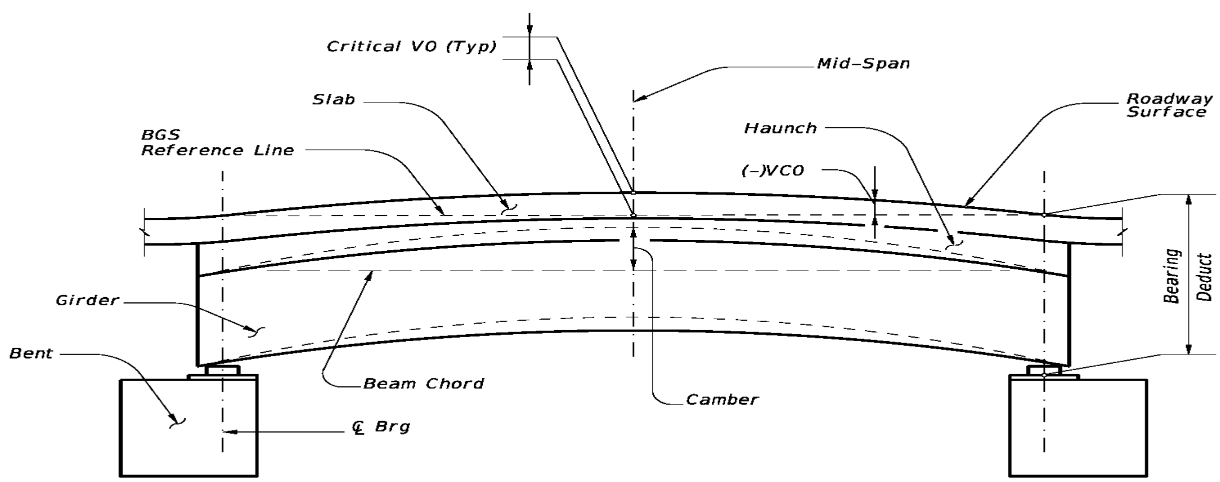


Figure A.1.4-2: All Haunch Components Working Together

A.1.5 Minimum and Maximum Haunch Values

The maximum haunch without reinforcing is 3½”.

The minimum haunch at the center of bearing is 2” to accommodate the thickened slab end.

The minimum haunch at mid-span is ½” to accommodate bedding strips for prestressed concrete panels.

A.2 Steps to Calculate Haunch

This section applies to simple geometric cases only.

Step 1

Execute a preliminary BGS run using a beam framing option from 1-10 in the FOPT card. On the BRNG card, input a "Depth Below the Reference Line", or bearing deduct, of zero. This gives the VCLR card output as the VCO defined above. Include a VCLR card for each span with the bridge alignment as the specified alignment.

Step 2

Examine the BGS output. The vertical ordinate (VO) will be given along the girder at as many segments as defined in the VCLR card. The first and last columns of each VCLR table are the ordinates at the center of bearings and should equal zero. The maximum magnitude VO is used in haunch calculations. The critical VO typically corresponds to the 0.5L column in the VCLR table as shown below. The sign convention shown in BGS should be used in the haunch calculations.

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PAGE 110
BRIDGE GEOMETRY SYSTEM (BGS) *** BRIDGE GEOMETRICS ***
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CONTROL SECTION COUNTY & HIGHWAY - TRAVIS CO US 183

VERTICAL CLEARANCE BETWEEN SPAN 14 OF ROADWAY H WITH ROADWAY H

	0.00 L	0.10 L	0.20 L	0.30 L	0.40 L	0.50 L	0.60 L	0.70 L	0.80 L	0.90 L	1.00 L
BEAM 1	0.00	0.02	0.04	0.05	0.06	0.06	0.06	0.05	0.04	0.02	0.00
BEAM 2	0.00	0.02	0.04	0.05	0.06	0.07	0.07	0.06	0.04	0.03	0.00
BEAM 3	0.00	0.02	0.04	0.06	0.07	0.07	0.07	0.06	0.05	0.03	0.00
BEAM 4	0.00	0.02	0.05	0.06	0.07	0.08	0.08	0.07	0.05	0.03	0.00
BEAM 5	0.00	0.03	0.05	0.07	0.08	0.08	0.08	0.07	0.06	0.03	0.00
BEAM 6	0.00	0.03	0.05	0.07	0.08	0.09	0.09	0.08	0.06	0.03	0.00

Figure A.2-1: BGS Output of VCLR Table

Step 3

Calculate the required minimum haunch at center of bearing at center of girder top flange that will work for all TxGirders in a span.

$$Haunch_{CL\,Brg,Req} = (C - 0.8\Delta_{DL}) + VCO + CSC + Haunch_{min,Req}$$

where:

C = Camber of the TxGirder (ft) (taken from PGSuper "TxDOT Summary Report" under "Camber and Deflections", the Factored Design Camber. Factored Design Camber includes girder self-weight deflection, camber at release, and camber due to creep.

Δ_{DL} = Absolute value of dead load deflection of TxGirder at midspan due to a cast-in-place slab (ft) (taken from PGSuper "TxDOT Summary Report" under "Camber and Deflection" for deflections of Deck and Diaphragms. NOTE: PGSuper gives dead load deflection in inches and feet. The dead load deflection will need to be in feet for the above equation.)

VCO = Maximum magnitude vertical clearance ordinate (ft) (taken from BGS VCLR output table; keep BGS sign convention).

CSC = Cross slope correction (ft) (this equation assumes a constant cross slope above the girder top flange).

$$= CS \times \frac{1}{2}(w_f)$$

where:

CS = Cross slope of slab above girder top flange (ft/ft).

w_f = Width of top flange (ft).

$Haunch_{min,Req}$ = Minimum required haunch measured at the least-haunch edge of the girder flange. Minimum haunch typically occurs at mid-span. However, with large crest curves and superelevation transitions, Minimum Haunch can occur anywhere along the beam.

Use the largest $\text{Haunch}_{\text{CL Brg,Req}}$ value for each girder in that span:

$$Haunch_{CL\ Brg} = max\{\text{Haunch}_{\text{CL Brg, Req}}\}$$

Round up to the nearest ¼″.

$Haunch_{CL\ Brg}$ is the haunch that will be provided at center of bearing for each TxGirder in that span.

Step 4

Calculate the theoretical minimum haunch along center of girder top flange.

$$Haunch_{min} = Haunch_{CL\ Brg} - (C - 0.8\Delta_{DL}) - VCO - CSC$$

Step 5

Calculate the slab dimensions at the center of bearing, "X"; the theoretical slab dimensions at mid-span, "Z"; and the depth from top of slab to bottom of girder at center of bearing, "Y", for each TxGirder in the span.

$$\text{"}X\text{"} = Haunch_{CL\ Brg} + t_s$$

$$\text{"}Y\text{"} = \text{"}X\text{"} + Girder\ Depth$$

$$\text{"}Z\text{"} = Haunch_{min} + t_s + CSC$$

where: t_s = Slab thickness

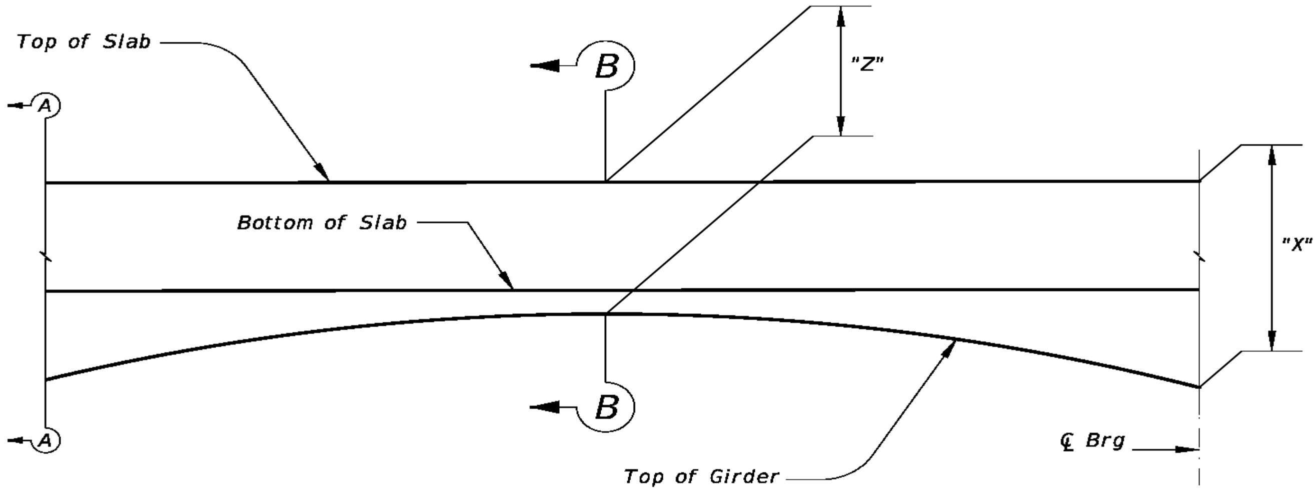

Figure A.2-2: "X" and "Z" Dimensions in Elevation View

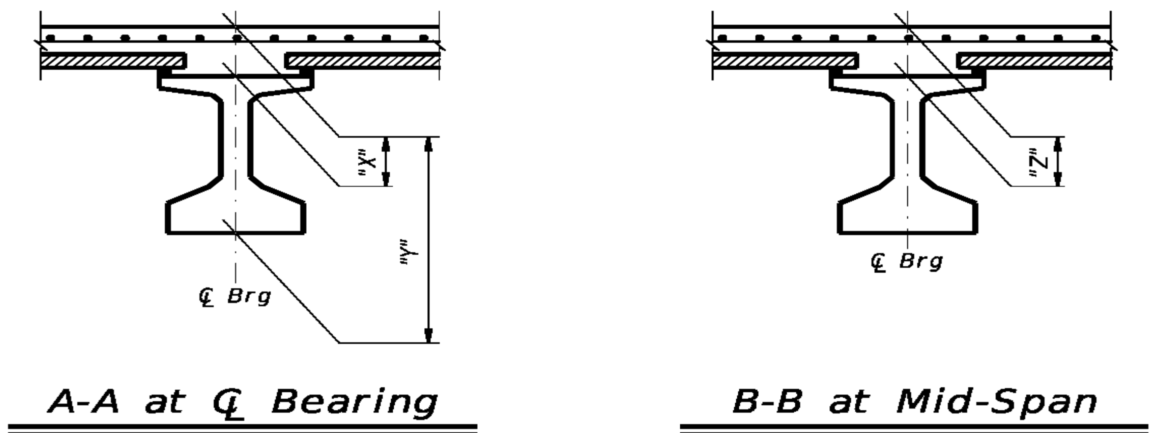

Figure A.2-3: "X", "Y", and "Z" Dimensions in Section View

NOTE: "Z" is a theoretical dimension. The true value depends on actual beam camber, which is difficult to predict.

Step 6

Calculate the required bearing deduct used in computing the final bearing seat elevations,

$$Bearing\ Deduct\ = \text{“}Y\text{”}\ +\ Bearing\ Pad\ Thickness.$$

If sole plates are required,

$$Bearing\ Deduct\ = \text{“}Y\text{”}\ +\ Bearing\ Pad\ Thickness\ +\ Sole\ Plate\ Thickness.$$

Round values to the nearest $^1/_8$”.

A.3 Vertical Curve Effects on Haunch

The three most common classes of sag and crest curve haunch effects are illustrated below. Figure A.3-1 shows a sag curve, which creates a minimum haunch at mid-span with a much larger haunch at the ends. Figure A.3-2 shows a large crest curve with respect to the girder camber. In this case, there is more haunch at mid-span than at the ends. Figure A.3-3 shows a small crest curve where there is less haunch at mid-span than at the ends.

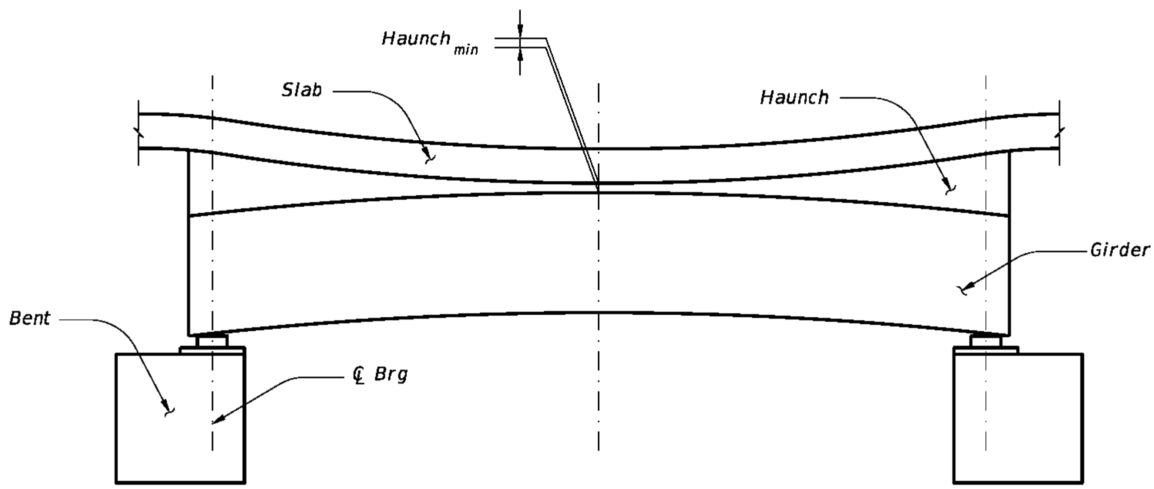

Figure A.3-1: Sag Curve Effect on Haunch

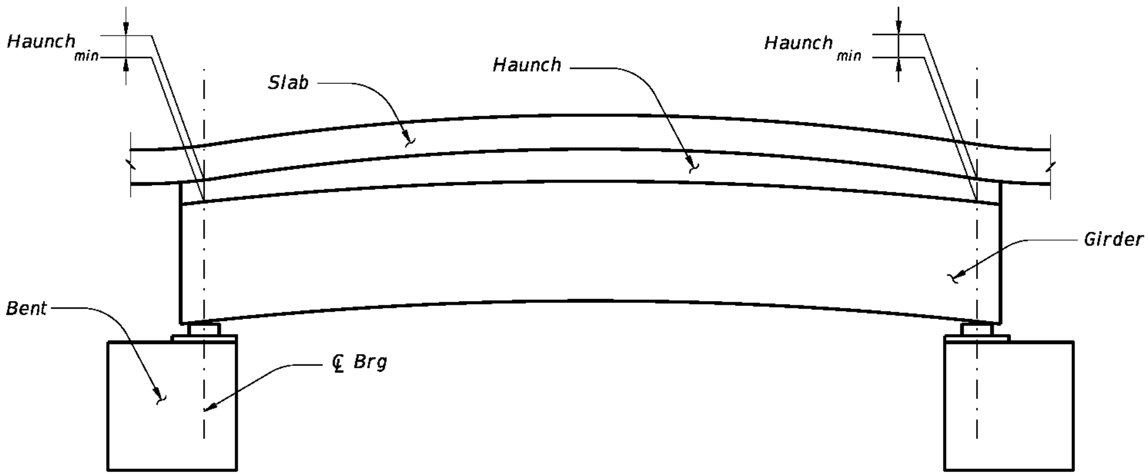

Figure A.3-2: Large Crest Curve Effect on Haunch

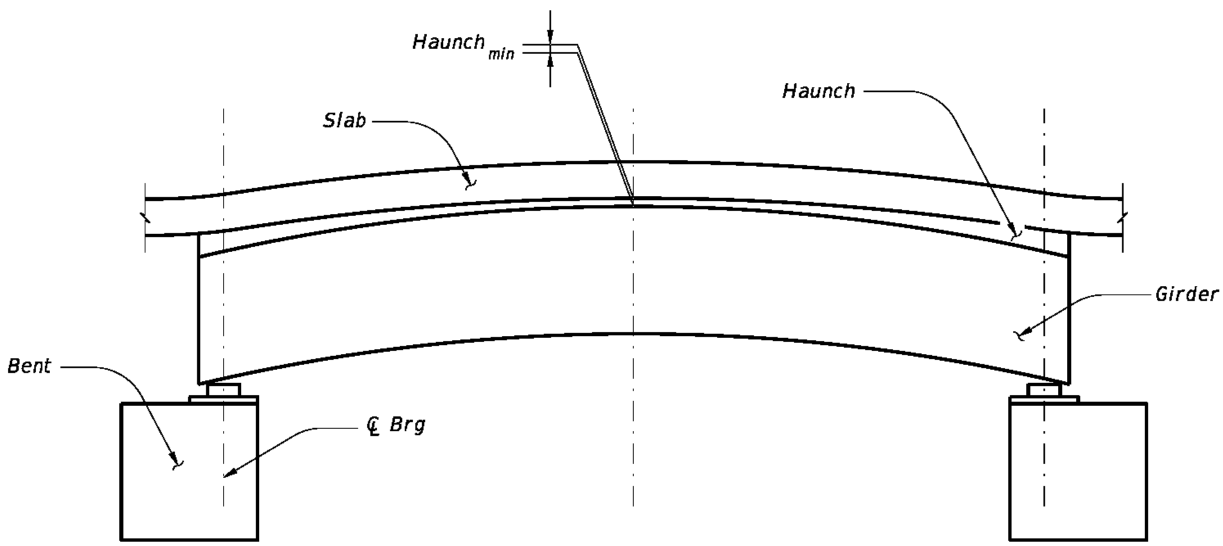

Figure A.3-3: Small Crest Curve Effect on Haunch

A.4 Superelevation Transition Effects on Haunch

Superelevation transitions can create unusual VCO patterns. The typical VCO pattern is a parabolic shape with a maximum or a minimum at mid-span. When there is a superelevation transition that starts or ends in a span, the VCLR table of BGS needs to be looked at more closely to determine if the haunch needs to be calculated at points other than mid-span. This also applies when a superelevation transition ends at a skewed bent because all the girders not along the profile grade line (PGL) will see the same effects as if the superelevation transition was terminated within the span. When the VCO output has an unusual pattern as shown below in Figure A.4-1, the haunch needs to be calculated at multiple points along the girders since the camber and dead load deflection vary along the girder. This means that it is incorrect to calculate the haunch using the critical VCO and the camber and dead load deflection from mid-span when the critical VCO does not happen at mid-span. When considering quarter points, dead load deflection can be calculated as 0.7123 x ΔDL and camber can be estimated as 0.7123 x C. Figure A.4-2 through Figure A.4-6 give a visual representation of the superelevation transition on the bridge. In general, the use of superelevation transitions increases the haunch to prevent encroachment of the girder into the slab.

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                         BRIDGE GEOMETRY SYSTEM (BGS)                *** BRIDGE GEOMETRICS ***
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CONTROL       SECTION                                               COUNTY & HIGHWAY -

VERTICAL CLEARANCE BETWEEN SPAN  1 OF ROADWAY H WITH ROADWAY H

          0.00 L  0.10 L  0.20 L  0.30 L  0.40 L  0.50 L  0.60 L  0.70 L  0.80 L  0.90 L  1.00 L

BEAM  1     0.00    0.00    0.34    0.44    0.54    0.63    0.73    0.82    0.92    1.01    0.00

BEAM  2     0.00    0.00    0.17    0.22    0.27    0.32    0.36    0.41    0.46    0.51    0.00

BEAM  3     0.00    0.00    0.00    0.00    0.00    0.00    0.00    0.00    0.00    0.00    0.00

BEAM  4     0.00    0.00    0.17    0.07   -0.03   -0.12   -0.22   -0.31   -0.41   -0.50    0.00

BEAM  5     0.00    0.00    0.33    0.14   -0.05   -0.24   -0.43   -0.63   -0.82   -1.01    0.00
```

Figure A.4-1: BGS VCLR Table Output for Superelevation Transition

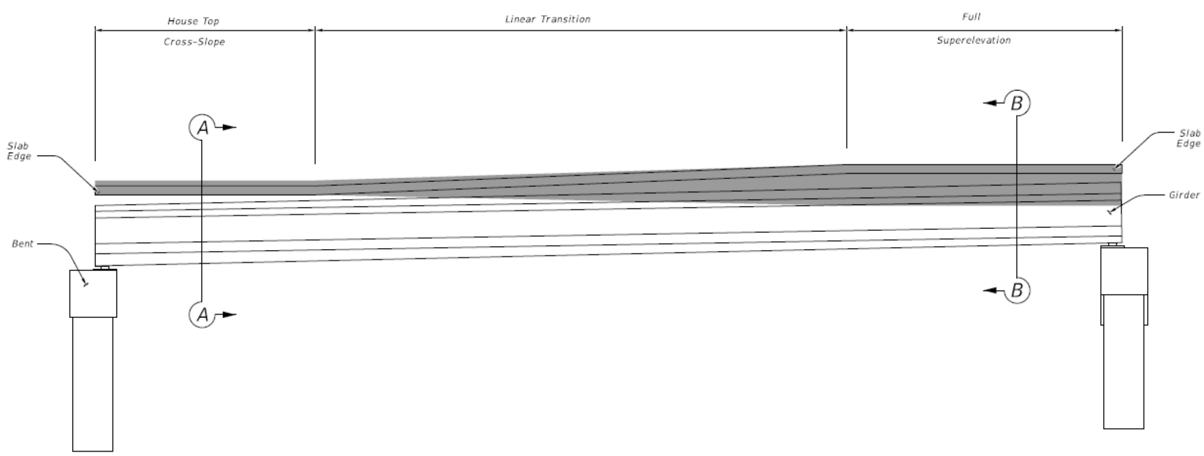


Figure A.4-2: Elevation View of a Superelevation Transition

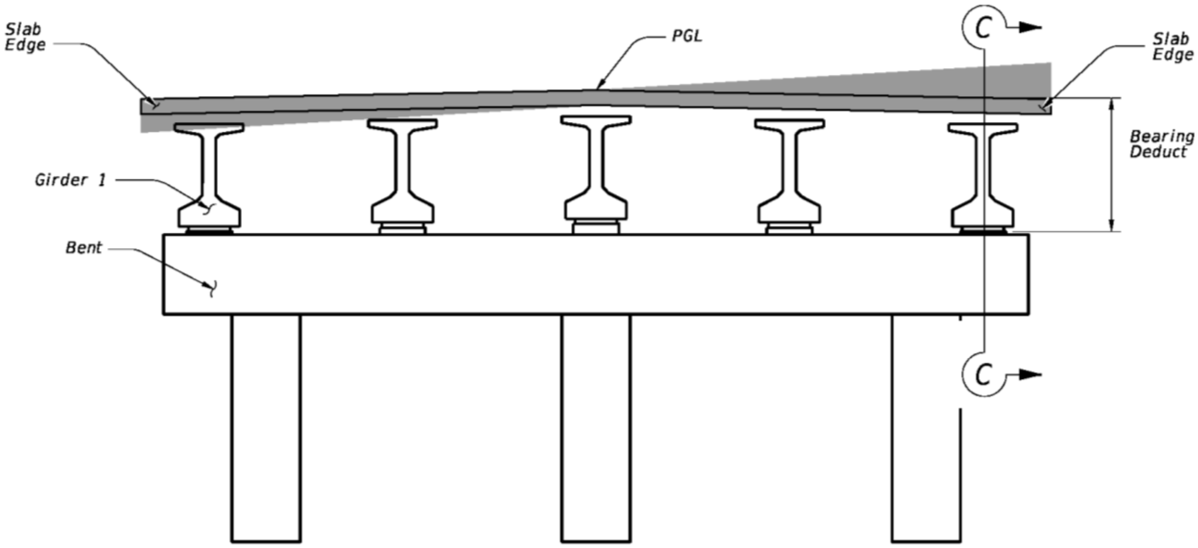


Figure A.4-3: Section A-A (House top cross-slope)

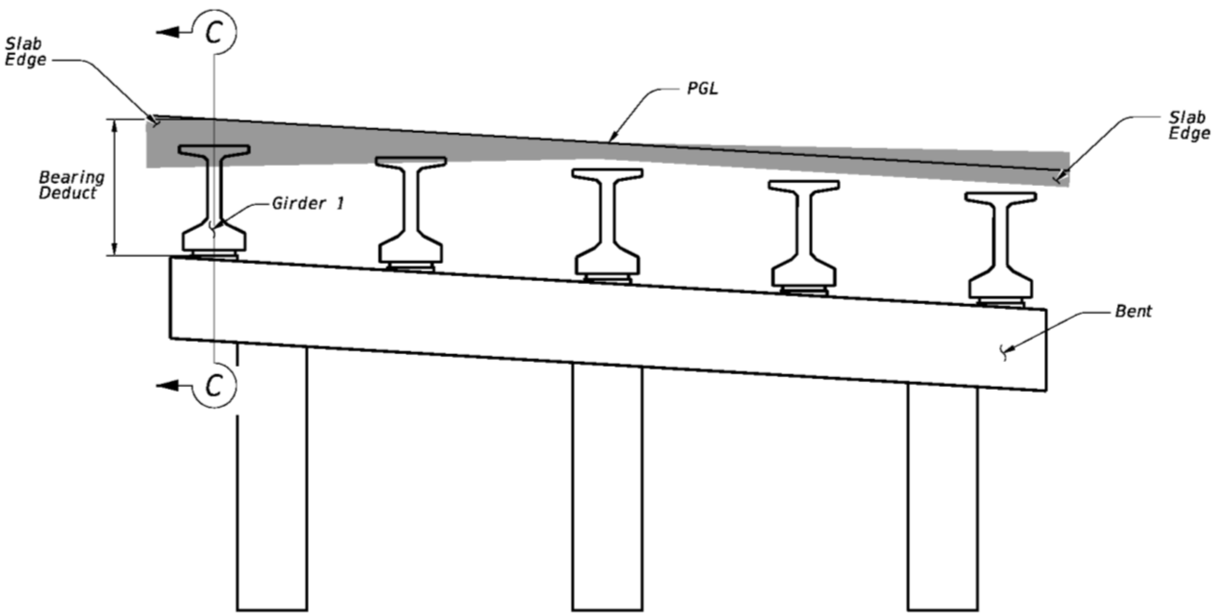


Figure A.4-4: Section B-B (Full superelevation)

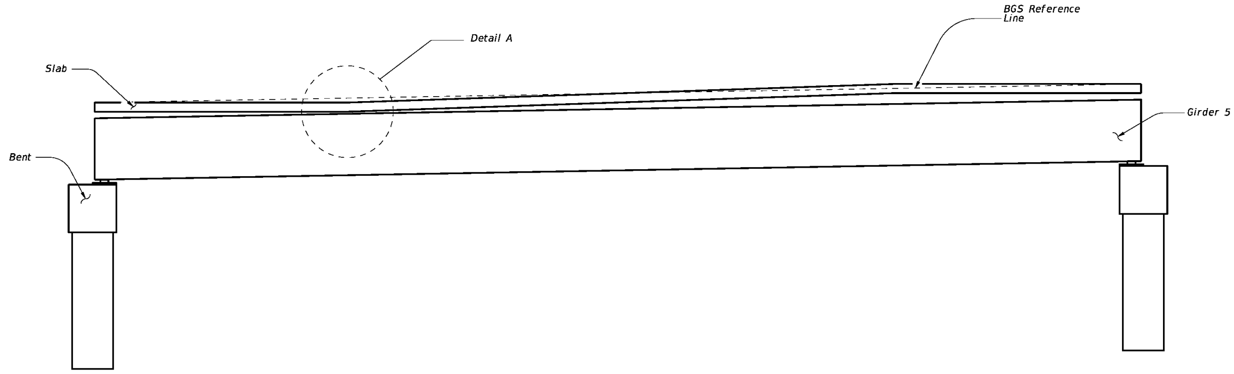
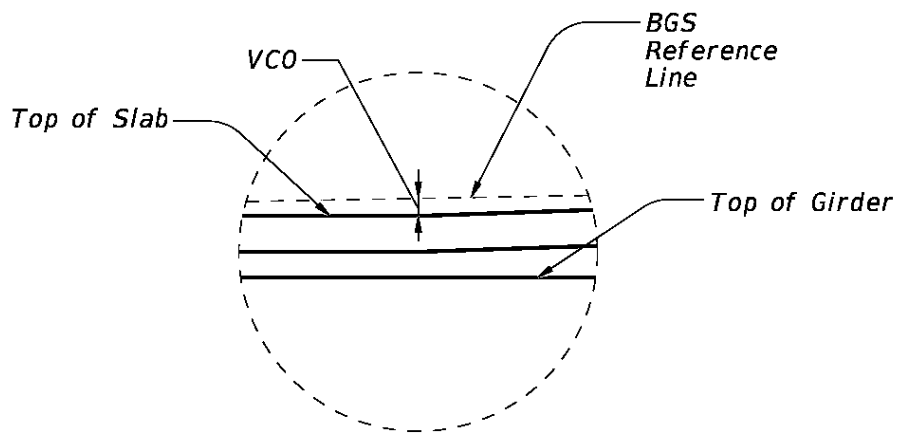


Figure A.4-5: Section C-C



Detail A

Figure A.4-6: Detail A

(Showing the kink that is created by a superelevation transition at mid span.)

Appendix B X-Beam Design Guide

B.1 Bearing Pad Taper Calculations for X-beams

The bearing seat for an X-beam is level perpendicular to the centerline of the bent but slopes along the centerline of bearing between the left and right bearing seat elevations. The bearing pad is oriented along the centerline of bearing. This configuration for the X-beam bearing allows the pad to taper in only one direction (perpendicular to the centerline of bearing). The amount of bearing pad taper depends on three factors:

- Grade of the X-beam
- Slope of the bearing seat (taper is along centerline bearing)
- Beam angle (angle between centerline beam and centerline bearing in plan view)

All X-beam projects should have a Bearing Pad Taper Report sheet that contains the various pad tapers for use by the bearing pad fabricator. This report summarizes the bearing pad taper perpendicular to the centerline of bearing for each beam bearing location. In BGS, the report is titled "Bearing Pad Taper – Fabricator's Report." The calculations that follow derive the formula for calculating the bearing pad taper perpendicular to centerline of bearing.

For purposes of developing the formulas for calculating bearing pad taper, the following sign convention will be used looking in the direction of increasing station numbers:

- Positive bearing seat slope is up and to the right
- Negative bearing seat slope is down and to the right
- Positive beam grade is up
- Negative beam grade is down
- Positive bearing pad taper is up

- Negative bearing pad taper is down

Case I

Beam Angle is $\theta < 90^\circ$ and Right Forward.

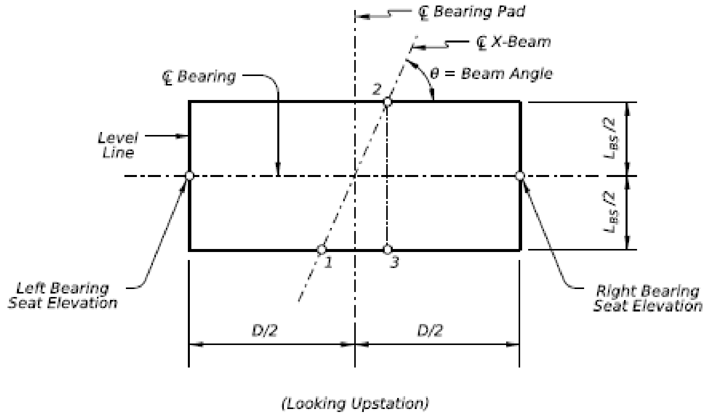


Figure B.1-1: Plan View of Bearing Seat with Right Forward Beam Angle

Perpendicular to the centerline of bearing, the pad taper is only a function of the bottom surface of the beam. The bearing seat is level in this direction, so the component of pad taper due to the top surface of the bearing seat is zero. Using $ELEV_1$, $ELEV_2$, and $ELEV_3$ at points 1, 2, and 3, respectively, at the bottom of X-beam, the equation for pad taper is:

$$TAPER = \frac{ELEV_2 - ELEV_3}{L_{BS}}$$

where:

$$ELEV_2 = ELEV_1 + BEAM\ GRADE\left(\frac{L_{BS}}{\sin\theta}\right)$$

$$ELEV_3 = ELEV_1 + SLOPE\left(\frac{L_{BS}}{\tan\theta}\right)$$

Substituting for $ELEV_2$ and $ELEV_3$, the equation for pad taper becomes:

$$TAPER = \frac{BEAM\ GRADE - SLOPE(\cos\theta)}{\sin\theta}$$

Case II

Angle is $\theta < 90°$ and Left Forward.

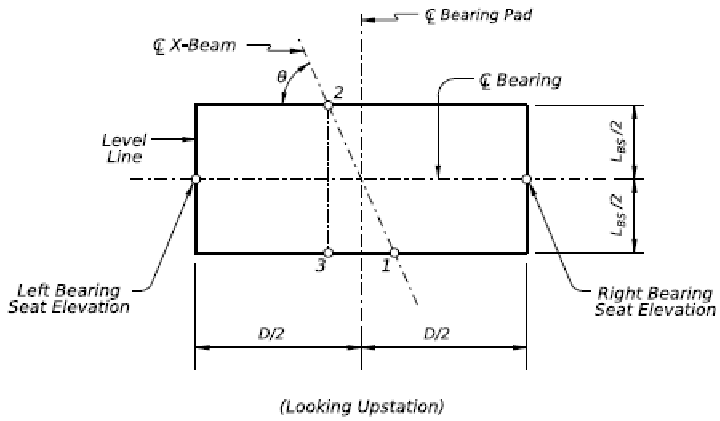


Figure B.1-2: Plan View of Bearing Seat with Left Forward Beam Angle

Using $ELEV_1$, $ELEV_2$, and $ELEV_3$ at points 1, 2, and 3, respectively, at the bottom of the X-beam, the equation for pad taper is:

$$TAPER = \frac{ELEV_2 - ELEV_3}{L_{BS}}$$

where:

$$ELEV_2 = ELEV_1 + BEAM\ GRADE\left(\frac{L_{BS}}{\sin\theta}\right)$$

$$ELEV_3 = ELEV_1 - SLOPE\left(\frac{L_{BS}}{\tan\theta}\right)$$

Substituting for $ELEV_2$and $ELEV_3$, the equation for pad taper becomes:

$$TAPER = \frac{BEAM\ GRADE + SLOPE(\cos\theta)}{\sin\theta}$$

Case III

Beam Angle is θ = 90°

Since the centerline of the X-beam is perpendicular to centerline of bearing, the component of bearing pad taper due to the bearing seat is zero, and the bearing pad taper is simply:

$$TAPER = BEAM\ GRADE$$

Summary

Defining β = beam angle as measured counterclockwise from centerline of bearing, the equation for the calculating bearing pad taper for any bearing location is as follows:

$$TAPER = \frac{BEAM\ GRADE - SLOPE(\cos\beta)}{\sin\beta}$$

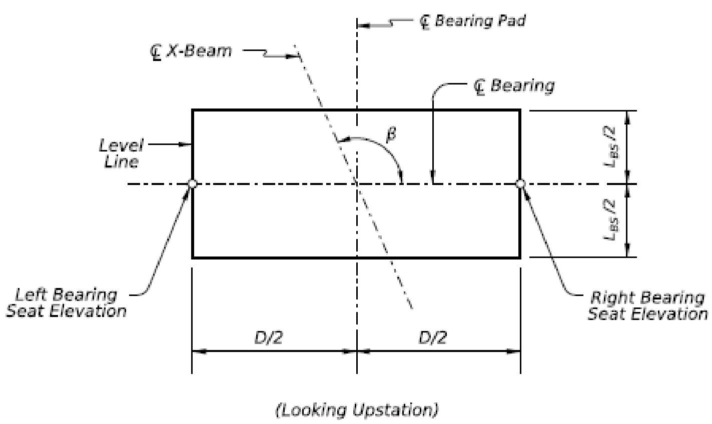


Figure B.1-3: Plan View of Bearing Seat

B.2 Haunch Calculations for X-beams

X-beams are placed at a cross-slope, unlike TxGirders. For spans with constant cross-slope and constant overall width, X-beam flanges will be parallel to the cross-slope of the roadway surface. For spans with more complicated geometry, such as varying cross-slope and/or varying overall width, X-beams will be at some cross-slope other than the cross-slope of the roadway surface. Each X-beam in a span is balanced in cross-slope from the back bearing to the forward bearing of the beam so that no torsion is introduced into the beam. Thus, the haunch at centerline of bearing for the left edge of the beam may be different than the haunch at right edge of the beam. Skewed beam end conditions can also contribute to a different haunch at centerline bearing for each edge of the beam. This difference can exist even with a constant cross-slope (i.e., it is due to the geometry of the roadway surface and not necessarily the balancing of the X-beam).

In terms of calculating the required haunch at centerline of bearing for an X-beam, the haunch for each edge of top flange of the X-beam must be calculated. Once the minimum haunch value is established, the maximum haunch at centerline of bearing on the opposite top edge of the X-beam can be calculated as well as the deduct value for computing bearing seat elevations. This section covers a suggested method of calculating the required haunch and the corresponding deduct values for X-beams.

When calculating the bearing seat elevations, be careful to apply the appropriate deduction at that elevation point - that is, the minimum deduction at the correct elevation point and the maximum deduction at the other elevation point. Typically, the minimum deduction and maximum deduction are each applied at diagonally opposite corners of a beam in plan view.

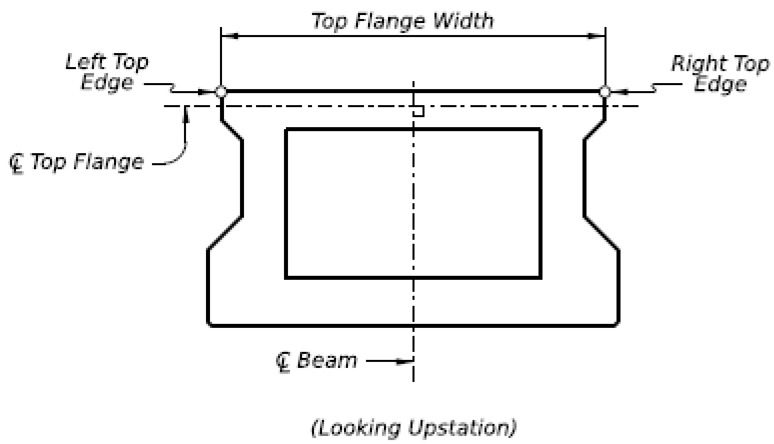


Figure B.2-1: X-beam Section View Definitions

Step 1

Execute a preliminary BGS run using beam framing option 20, 21, or 22 to calculate the vertical curve component of the haunch. These options were originally developed for U-beam spans. Each option assures that no torsion is allowed in the beams by balancing the cross-slopes at each end of each beam. Each beam is balanced individually. Option 20 is for beams equally spaced between outside beam lines. Option 21 is for beams defined as parallel to the outside beams. Option 22 is used when all beam lines are defined individually.

On the BRNG card, input the section depth as zero and the pedestal width equal to the top flange width of the X-beam (43.75 inches for 4XB and 55.75 inches for 5XB). Add the letter “P” in column 80 of the BRNG card. This instructs BGS to keep the top flange width dimension perpendicular to the centerline of X-beam. Thus, for skewed beam end conditions, BGS will account for the skew at that end of the X-beam and give the corresponding vertical clearance ordinates (VCO) at the centerline of bearing for the left and right edges of the top flange (See Figure B.2-2). Also, include a VCLR card for each span with the bridge alignment as the specified alignment.

Step 2

Three lines of vertical clearance ordinates (VCO) will be generated for every X-beam: the VCOs along the left top edge, centerline, and right top edge of X-beam. The first and last columns of each VCO table are the ordinates at centerline of back bearing and forward bearing, respectively. One or both VCO values at the left and right top edge of the X-beam at centerline of bearing will be zero. A VCO of zero indicates that the top edge of the X-beam is matched with the elevation of the top of slab at that point. Thus, the vertical placement is controlled by these “corners” of the X-beam that have VCO of zero.

The corner opposite to the controlling corner at the centerline of bearing will either have a zero or negative VCO. Its value depends on the bridge geometry and/or balancing of the X-beam. A zero value for the VCO at the dependent corner means that at that point the top of the X-beam is also matched with the elevation of the top of slab. However, a negative VCO value at the dependent corner means that at that point the top edge of X-beam is below the elevation at top of slab. This negative VCO is the difference in haunch at centerline of bearing from the left top edge of beam to the right top edge of beam due to bridge geometry and/or balancing of the X-beam (Figure B.2-2).

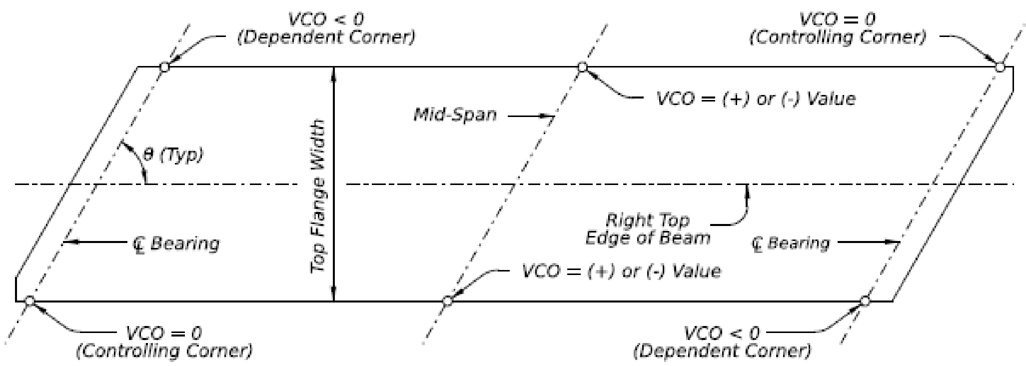


Figure B.2-2: Example Plan View of the Vertical Clearance Ordinates for X-beam

Figure B.2-3 illustrates the VCOs produced by BGS for the left and right top edges of the X-beam when framing a span with a crest vertical curve and a varying cross-slope along the span. It is shown only to help visualize a possible scenario of VCOs produced by BGS.

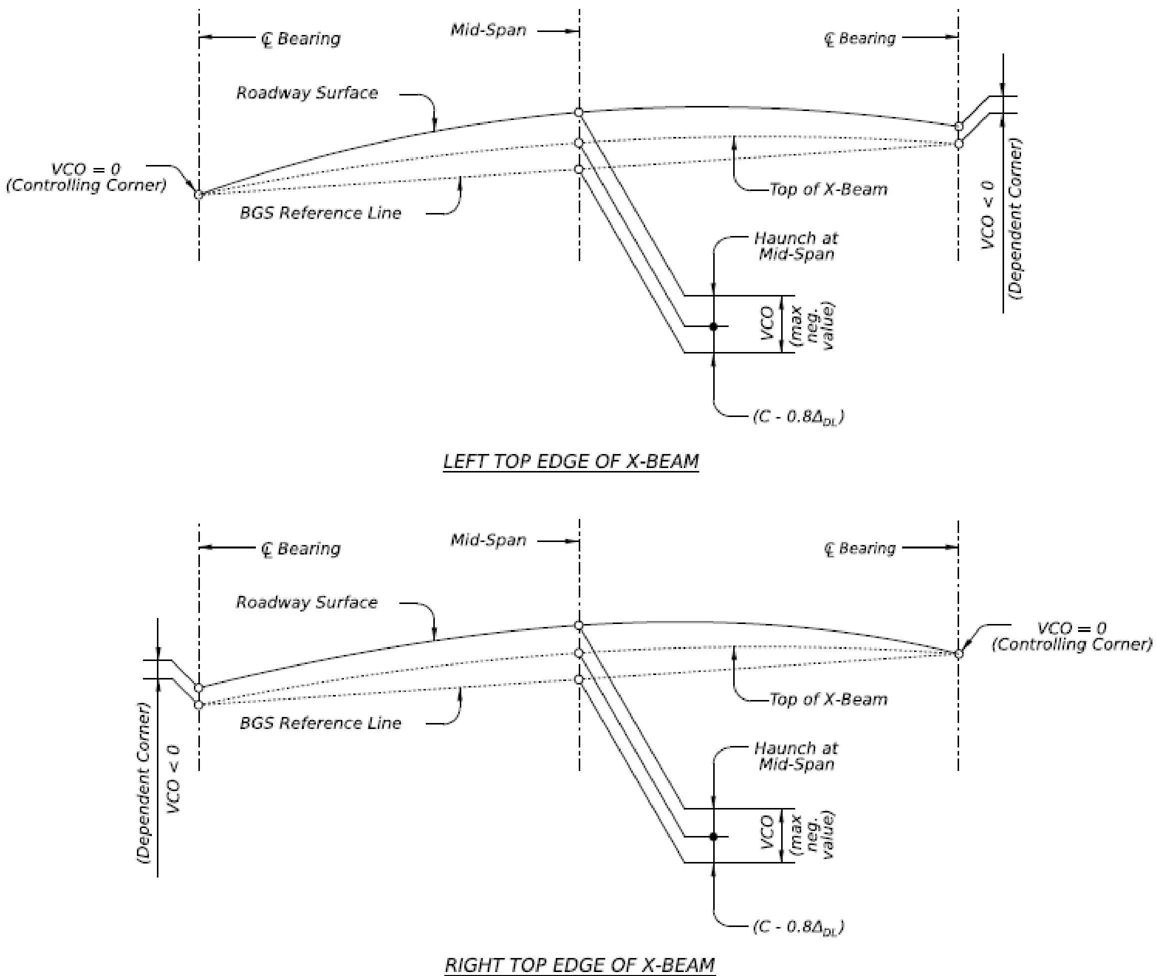


Figure B.2-3: Example Vertical Profile at Edges of X-beam

When inputting the top flange width of the X-beam, b_f, on the BRNG card, the VCLR command calculates the VCOs at an offset distance of $b_f/2$ from the BGS beam line (See B.2-4). The standard convention for defining the BGS beam line is a vertical line at a point coinciding with the centerline of the bottom of the bearing pad. Thus, for X-beams at a cross-slope, the beam rotates about this point. This rotation of the X-beam shifts the top flange of the beam transversely with respect to the BGS beam line. BGS makes no adjustment for the rotation-induced transverse movement and, therefore, VCO is not calculated exactly at the outside edge of the top flange.

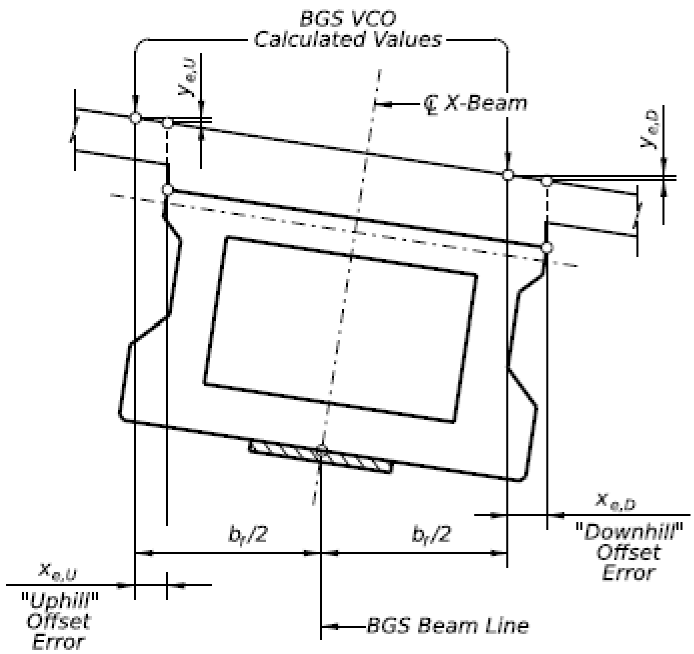


Figure B.2-4: Example of X-beam Geometry as per BGS

The offset error may be computed as follows, but it is usually negligible.

$$\psi_1 = h_u \tan(\alpha)$$

$$\psi_u = \frac{b_f}{2\cos(\alpha)} + \psi_1 - \frac{b_f}{2} \begin{Bmatrix} X_{e,U} = \psi_u \cos\alpha \\ Y_{e,U} = \psi_u \sin\alpha \end{Bmatrix}$$

$$\psi_D = \frac{b_f}{2} + \psi_1 - \frac{b_f}{2\cos(\alpha)} \begin{Bmatrix} X_{e,D} = \psi_D \cos\alpha \\ Y_{e,D} = \psi_D \sin\alpha \end{Bmatrix}$$

where:

α = cross slope angle in radians = arctan(cross slope).

b_f = top flange width.

h_u = height of the X-beam.

Table B.2-1: BGS Offset Errors for Common Cross Slopes (4XB40)

Common Cross Slopes	α (radians)	$X_{e,U}$ (in)	$Y_{e,U}$ (in)	$X_{e,D}$ (in)	$Y_{e,D}$ (in)
⅛″ per 1'	0.0104	0.42	0.00	0.42	0.00
1.5%	0.0150	0.60	0.01	0.60	0.01
2%	0.0200	0.80	0.02	0.80	0.02

Table B.2-2: BGS Offset Errors for Common Cross Slopes (5XB40)

Common Cross Slopes	α (radians)	$X_{e,U}$ (in)	$Y_{e,U}$ (in)	$X_{e,D}$ (in)	$Y_{e,D}$ (in)
⅛″ per 1'	0.0104	0.42	0.00	0.42	0.00
1.5%	0.0150	0.60	0.01	0.60	0.01
2%	0.0200	0.81	0.02	0.79	0.02

Step 3

Calculate the required minimum haunch at centerline bearing that will work for all X-beams in a span. Start by calculating the required minimum haunch at centerline of bearing for both the left and right top edges of each X-beam in that span. For each side of the beam, work from the controlling corner and use the entire maximum vertical clearance ordinate (VCO) on that edge in your haunch calculation. Do not be concerned with the VCO value at the dependent corner for each side, because that value affects only the maximum haunch (see Step 4 below), not the minimum haunch.

Also, a note to users who obtain camber and dead load deflection values from PGSuper. The reduction coefficients presented in this section are stored within PGSuper and can be utilized by its haunch algorithm.

Follow these steps to obtain the raw values (which are required when using the equations in this document) from the TxDOT Summary Report in PGSuper:

- Camber: Use *Design Camber* under "Camber and Deflections".
- Δ_{DL}: Use *Slab and Diaphragms* deflection under "Camber and Deflections".

A positive VCO means the top of beam is above the top of slab at that point, while a negative ordinate means the top of beam is below the top of slab at that point. Keeping the sign convention used by BGS, the required minimum haunch values at centerline of bearing for each X-beam are as follows:

For Controlling Corners along the Left Top Edge:

$$Haunch_{CL\ Brg,Req(L)} = (C - 0.8\Delta_{DL}) + VCO_{\max(L)} + Haunch_{min,Req} + y_e$$

For Controlling Corners along the Right Top Edge:

$$Haunch_{CL\ Brg,Req(R)} = (C - 0.8\Delta_{DL}) + VCO_{\max(R)} + Haunch_{min,Req} + y_e$$

where:

$VCO_{\max(L)}$ = Maximum VCO, left top edge (usually at mid-span)

$VCO_{\max(R)}$ = Maximum VCO, right top edge (usually at mid-span)

C = Camber of X-beam

Δ_{DL} = Dead load deflection of X-beam due to slab only (no haunch)

$Haunch_{min,Req}$ = The minimum required haunch (usually at mid-span) is ½"

y_e = Vertical BGS Offset Error; this value is usually small and negligible. If the user wishes to account for this error, assign a positive value for the “downhill” haunch and a negative value for the “uphill” haunch.

Next, calculate the largest haunch required at a controlling corner for that span:

$$Haunch_{CL\ Brg(CC)} = max\{Haunch_{CL\ Brg,Req(L)}, Haunch_{CL\ Brg,Req(R)}\}$$

Round up to the nearest ¼”.

This haunch value will be the haunch at all controlling corners for each X-beam in that span. Now, calculate the theoretical minimum provided haunch on each side:

$$Haunch_{\min(L)} = Haunch_{CL\ Brg(CC)} - (C - 0.8\Delta_{DL}) - VCO_{\max(L)} - ye$$

$$Haunch_{\min(R)} = Haunch_{CL\ Brg(CC)} - (C - 0.8\Delta_{DL}) - VCO_{\max(R)} - ye$$

Step 4

Calculate the corresponding maximum haunches at centerline of bearing. The maximum haunches at centerline of bearing occur at the dependent corners of each X-beam. These maximum haunches may vary between X-beams in a span but typically will not vary for the same X-beam. The maximum haunches at centerline of bearing for each X-beam in a span are:

Left Top Edge:

$$Haunch_{CL\ Brg(LDC)} = Haunch_{CL\ Brg(CC)} - VCO_{LDC}$$

Right Top Edge:

$$Haunch_{CL\ Brg(RDC)} = Haunch_{CL\ Brg(CC)} - VCO_{RDC}$$

where:

VCO_{LDC} = Vertical clearance ordinate value, dependent left top corner

VCO_{RDC} = Vertical clearance ordinate value, dependent right top corner

Step 5

Calculate the slab dimensions at centerline of bearing, X_{min} and X_{max}, and the theoretical slab dimensions at mid-span, Z_L and Z_R, for each X-beam in the span (See Figure B.2-5).

The equations are:

$$X_{min} = Haunch_{CL\ Brg(CC)} + slab\ thickness$$

$$X_{max} = Haunch_{CL\ Brg(LDC\ or\ RDC)} + slab\ thickness$$

$$Z_L = Haunch_{min(L)} + slab\ thickness$$

$$Z_R = Haunch_{min(R)} + slab\ thickness$$

Again, X_{min} is the section depth at all controlling corners of the beam while X_{max} is the section depth at all dependent corners of the beam (any difference in X_{max} for each dependent corner of an individual beam should be negligible).

TABLE OF SECTION DEPTHS					
Span No.	Beam No.	"X_A" at ℄ Bearing	"X_B" at ℄ Bearing	① "Z_L" at ℄ Bearing	①"Z_R" at ℄ Bearing

① Theoretical dimension

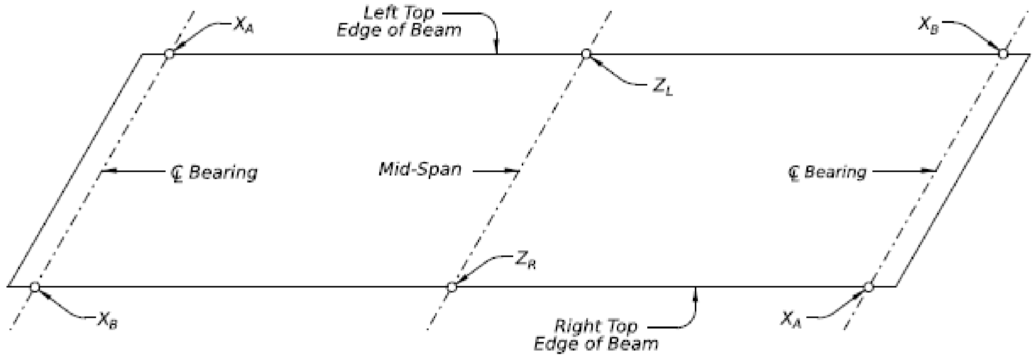


Figure B.2-5: Section Depth Information on Production Drawings

Figure B.2-5 shows a typical plan view and table that can be used on production drawings to describe the depth and location of the X and Z values. In order to use a single and generic detail, the “min” and “max” designation is changed to an open convention using the letters “A” and “B”. As a result, X_A and X_B can be either the X_{min} or X_{max} values.

Step 6

Calculate the required deduct at the specific bearing location to use in computing the final bearing seat elevations. The XBEB standard sheets provide bearing seat build-up widths (pedestal widths in BGS terminology) for the 5XB and 4XB beams with the standard end conditions. The widths listed depend on the beam angle and are adequate for up to two bearing pads. Only one pedestal width can be input per BRNG card, use the pedestal width for the X-beam with the smallest beam angle at that bearing location. Omit the letter “P” in column 80 of the BRNG card so that BGS applies the width along the centerline of bearing.

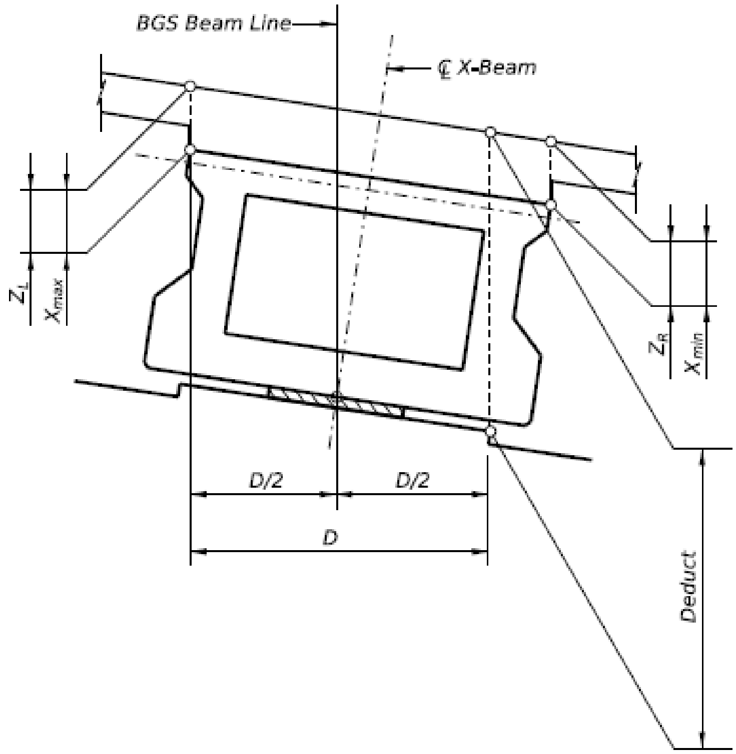


Figure B.2-6: Location of Deduct for Final Bearing Seat Elevations

The required deduct for calculating bearing seat elevations needs to be the deduct at the edge of the bearing seat (see Figure B.2-6). This deduct can be obtained by interpolating between the values X_{min} and X_{max} for each X-beam using the beam angle, the top flange width of the beam, and the chosen bearing seat width. The largest calculated deduct at that bearing location should be used to compute the final bearing seat elevations for all the X-beams at that bearing location. The difference between the calculated deducts at a given bearing location should be negligible but check the worst-case span initially to see if the difference is large enough to warrant adjusting the deduct.

The deduct for the calculation of final bearing seat elevations is:

$$Deduct = \left(\frac{X_{max} - X_{min}}{b_f \div sin(\theta)}\right)\left(\frac{b_f - D}{2}\right) + X_{min} + \frac{Beam\ Depth}{cos(\alpha)} + \frac{Bearing\ Pad\ Thickness}{cos(\alpha)}$$

where:

α = cross slope angle = arctan(cross slope).

b_f = top flange width.

θ = beam angle.

D = bearing seat width.

Note: The beam angle and cross slope can usually be ignored because of negligible difference in the final deduct amount.

Summary

The required haunch at centerline of bearing for the left and right top edges of the X-beam should always be calculated working from the controlling corner for that side. This is done because the vertical clearance ordinate for the dependent corner is “built-in” to the geometry for the beam and bridge. We cannot use that value in determining our haunch because the vertical clearance ordinate at the dependent corner is always present, i.e., we cannot adjust the beam vertically to reduce that dimension.

Basically, the controlling corners will have the minimum haunch at centerline of bearing. The dependent corners will have the maximum haunch at centerline of bearing. The difference is the vertical clearance ordinate value at the dependent corner.

Incidentally, because the X-beam is at an average cross-slope, the haunches at centerline of bearing for the back end of the beam for the left and right edges will typically be reversed at the forward end of the beam.

At mid-span, the theoretical haunch value for the left and right top edges of each X-beam will be the same value if the roadway surface cross-slope is constant or transitions at a constant rate over the entire length of the span and the beam spacing remains constant in the span. For any other case, the theoretical haunch at mid-span may be different for the left and right top edges of each X-beam. Also note that, as with any beam type, the minimum haunch does not always occur at mid-span. With large crest curves and superelevation transitions, minimum haunch can occur anywhere along the beam.

Appendix C Steel Twin Tub Girder System Redundancy Simplified Method Guide

C.1 Overview

One of the most accurate ways to assess the performance of a steel twin tub Girder Bridge in the event that one of the tension flanges fractures is through finite-element modeling. This type of modeling requires a substantial amount of time to develop and analyze. Simplified procedures for evaluating the redundancy of steel twin tub girder bridges were developed on the basis of behavior observed during a series of full-scale tests, which were part of a TxDOT research project, *Modeling the Response of Fracture Critical Steel Box-Girder Bridges*, (Barnard et al., 2010). The following gives an overview of the simplified method that was developed to evaluate twin tub bridges for system redundancy in lieu of finite element modeling.

C.1.1 Criteria for Use of Simplified Method

In order to use the simplified method, the following criteria must be met:

- Spans do not exceed 250 feet.
- Supports are skewed no more than 20 degrees.
- Horizontal curvature is greater than R = 700 feet.
- The Engineer ascertains that the use of an approximate method is adequate.

C.1.2 Assumptions

Refer Section 5.6.3 for assumptions, assumed fracture location, worst case loading condition, and live load positioning.

C.2 Simplified Method Procedure Outline

Design the bridge as normally done with the following exceptions:

- Design for Strength Limit State using a Redundancy Factor, η_R = 1.05.
- Design for Infinite Fatigue life for Fatigue and Fracture Limit State.

Then design the bridge for member failure under Extreme Event III.

Step 1:

Assume one girder is fractured within the critical span under consideration.

Fracture the girder at the bottom flange in tension and webs attached to that flange. Assume the other girder in the span under consideration is intact. For continuous units, assume that both girders are still intact in adjacent spans.

The location of the fracture within the span is assumed to be at the maximum factored tensile stress in the bottom flange determined using Strength I load combination.

The fractured girder should be the girder that would result in the worst loading scenario.

Step 2:

Calculate the transmitted load to the intact girder.

It is assumed that just prior to the fracture event, the girder that will fracture is carrying 50% of the total dead load of the bridge and all the live load, due to the position of the live load. Once the fracture occurs, the slab must transfer the entire load the fractured girder was carrying to the intact girder via the bridge slab. Therefore, the intact girder will now be carrying 100% of the dead load of the bridge and the entire live load.

$$F = (1.2)\left((L)\left(W_{girder} + \frac{W_{deck}}{2} + \frac{W_{ralings}}{2}\right) + W_{LL}\right)$$

where:

F = Transmitted load to intact girder (kips).

W_{girder} = Weight of one steel tub girder plus weight of diaphragms and stiffeners, etc. (kips).

W_{deck} = Concrete deck and haunches weight (kips).

$W_{ralings}$ = Total Railing weight (kips).

W_{LL} = Live Load (kips).

1.2 = Dynamic Increase Factor, DIF.

Step 3:

Calculate the maximum moment on the bridge.

a. Maximum moment due to dead load:

$$M_{DL} = \left(\frac{L^2}{8}\right)\left(2W_{girder} + W_{deck} + W_{railings}\right)$$

b. Maximum moment due to live load:

Position the HL-93 live load, including truck and lane load on the bridge deck directly above the fracture location.

The number, width, and location of design lanes is taken as the number, width, and location of striped traffic lanes on the bridge.

The live load is notional and meant to capture an envelope.

The impact factor, IM, is zeroed out for the fracture event

Step 4:

Calculate the bending capacity demand on the intact girder under Extreme Event III, according to Section 3.8.1:

a. Φ, Resistance Factor = 1.0
b. DL Load Factor = 1.10
c. LL Load Factor = 1.10
d. DIF, Dynamic Increase Factor = 1.2
e. $M_{EEIII} = (1.2)(1.10M_{DL} + 1.10M_{LL})$
   where: M_{EEIII} = Moment of member at failure under Extreme Event III

Step 5:

Calculate the plastic moment capacity, M_P, of the intact girder to determine if it has sufficient capacity to sustain the total live load and total dead load on the bridge.

$M_p \geq M_{EEIII}$

Step 6:

Check the bending and shear capacity of the concrete deck to ensure adequacy to resist the moment and shear produced by the unsupported load of the fractured girder.

Positive Moment Capacity, M_n^+ of Concrete Deck

The assumed strain and stress gradients at positive moment regions are shown in Figure C.2-1.

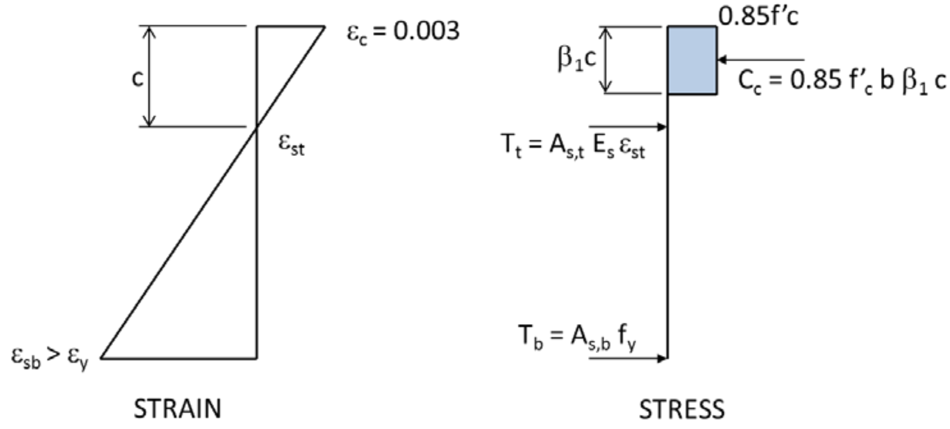


Figure C.2-1: Strain and stress gradients at positive moment regions

Take the moments about the neutral axis to solve for the nominal moment capacity.

Negative Moment Capacity, M_n^- of Concrete Deck

The assumed strain and stress gradients at negative moment regions are shown in Figure C.2-2.

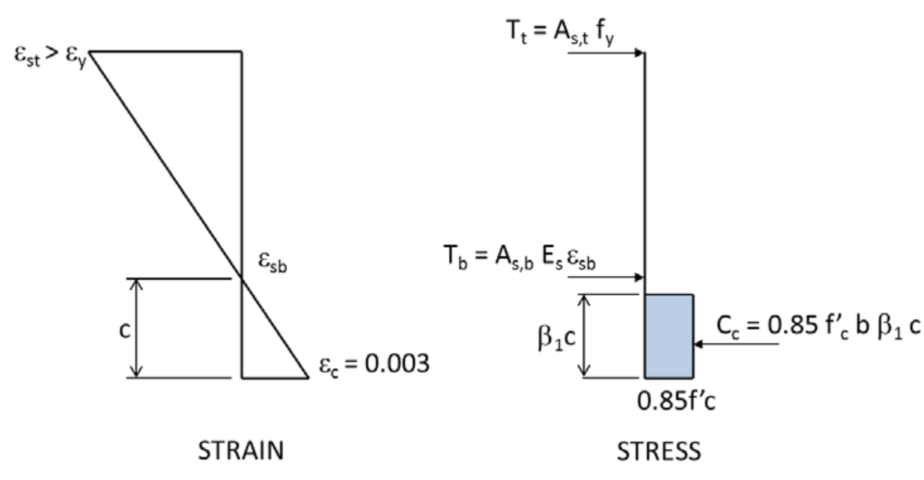


Figure C.2-2: Strain and stress gradients for negative moment regions

Take the moments about the neutral axis to solve for the nominal moment capacity.

Bending and Shear Capacity Check of Concrete Deck:

The deflected shape of the concrete deck and the bending moment diagram, if the shear studs have adequate tensile capacity, is shown below:

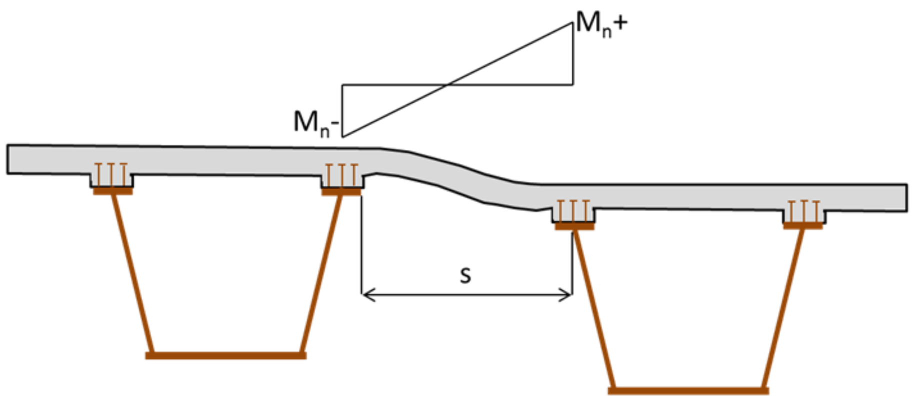


Figure C.2-3: Deflected shape and moment diagram before any failure of shear studs

The shear associated with the plastic deck mechanism is:

$$V_{PDM} = \frac{(M_n^+ + M_n^-)}{s}$$

where:

M_n^+ = Positive Moment Capacity of Concrete Deck (k-ft)

M_n^- = Negative Moment Capacity of Concrete Deck (k-ft)

s = Distance between the mid-width of the fractured girder's interior top flange and edge of the interior top flange of the intact girder (ft)

Calculate the shear capacity, V_c, using the American Concrete Institute (ACI) equation below and based on a 12-inch wide transverse deck section.

$$V_c = 2\sqrt{f_c'}bd$$

where:

f_c' = Compressive strength of concrete for use in design (psi).

b = Width of compression face of member (in).

d = Effective depth of member, distance from extreme compression fiber to centroid of longitudinal tension reinforcement (in).

The maximum shear capacity is taken as the smaller of the shear corresponding to a plastic moment mechanism in the deck and the shear capacity of the deck. Use the controlling shear, lesser of V_{PDM} and V_c, to calculate the total length, L_M, required to transfer the transmitted load, F.

$$L_M = \frac{F}{Controlling\ Shear}$$

Step 7:

Check the behavior of the shear studs.

The shear studs connecting the deck to the fractured girder must have sufficient tension capacity to develop the plastic beam mechanism in the bridge deck. The shear force on the studs in the fractured girder is assumed to be zero since it is assumed that no load is being carried by the fractured girder. The shear force on the studs in the intact girder, which are assumed to be not subject to tension, must satisfy <Article 6.10.10.4> to ensure composite action between the intact girder and the slab. Due to the inherent conservatism in the simplified method, and observations from laboratory testing of actual girders, it is reasonable to neglect the tension in the studs over the intact girder.

Determine the tensile strength of the shear stud group. The tensile capacity of the shear stud group can be obtained by using the modified ACI equations (ACI 2019 17.4.2.2), which are also included in *The Tensile Capacity of Welded Shear Studs* (Mouras, et.al. 2008). The ACI equations serve as a good basis for predicting the tensile strength of shear studs, but it does not adequately address these connections, especially when the detail has a haunch, therefore they must be modified for haunch.

$$N_b = k_c \sqrt{f_c'} h_h^{1.5}$$

where:

N_b = Concrete cone breakout strength of a single isolated stud in a continuous piece of cracked concrete (lbs).

k_c = 24 for cast-in-place shear studs.

f_c' = Specified concrete compressive strength (psi).

h_h= Effective height of the stud above the top of the haunch (in).

$$h_h = h_{ef} - d_h \geq \frac{w_h}{3}$$

h_{ef} = Effective height of shear stud in concrete, which is equal to the length of stud excluding the height of the stud head (in).

d_h = Haunch height (in).

w_h = Width of haunch perpendicular to bridge span axis.

To determine whether the shear studs pull out or a hinge is formed in the concrete deck, calculate N_{cbg}, which is the design concrete breakout strength of a stud or group of studs.

$$N_{cbg} = \frac{A_{NC}}{A_{NCO}} \psi_{g,N} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} N_b$$

where:

N_{cbg} = Design concrete breakout strength of a stud or group of studs (kips).

A_{NC} = Projected concrete cone failure area of a stud group (in^2).

$A_{NC} = 3h_{ef}w_h$.

If no haunch is present, the group failure cone area is defined by the total area covered by overlapping single stud failure cone areas. If a haunch is present, the sides of the haunch are assumed to act as an edge in confining the size of the failure cone to the haunch region. See Figure C.2-4 and Figure C.2-5 for more information.

A_{NCO} = Projected concrete cone failure area of a single stud in continuous concrete (in^2) See Figure C.2-4 and for Figure C.2-5 more information.

$A_{NCO} = 9h_h^2$.

$\psi_{g,N}$ = Group effect modification factor for studs on a bridge girder.

= 1.0 for 1 stud.

= 0.95 for 2 studs spaced transversely.

= 0.90 for 3 studs spaced transversely.

= 0.80 for studs spaced longitudinally ≤ $3h_{ef}$.

$\psi_{ec,N}$ = Eccentric load modification factor.

$$\psi_{ec,N} = 1 + \frac{2e'_N}{3h_h} \leq 1.0$$

where:

e'_N = Eccentricity of resultant stud tensile load.

$\psi_{ed,N}$ = Edge distance modification factor.

= 1.0 for $c_{a,min}$ ≥ 1.5 h_h

$= 0.7 + 0.3\frac{c_{a,min}}{1.5h_h}\ for\ c_{a,min} < 1.5h_h$

$c_{a,min}$= smallest edge distance measured from center of stud to the edge of concrete (in).

$\psi_{c,N}$ = Cracked concrete modification factor.

= 1.0, Cracked or no haunch.

= 1.25, Uncracked or with haunch.

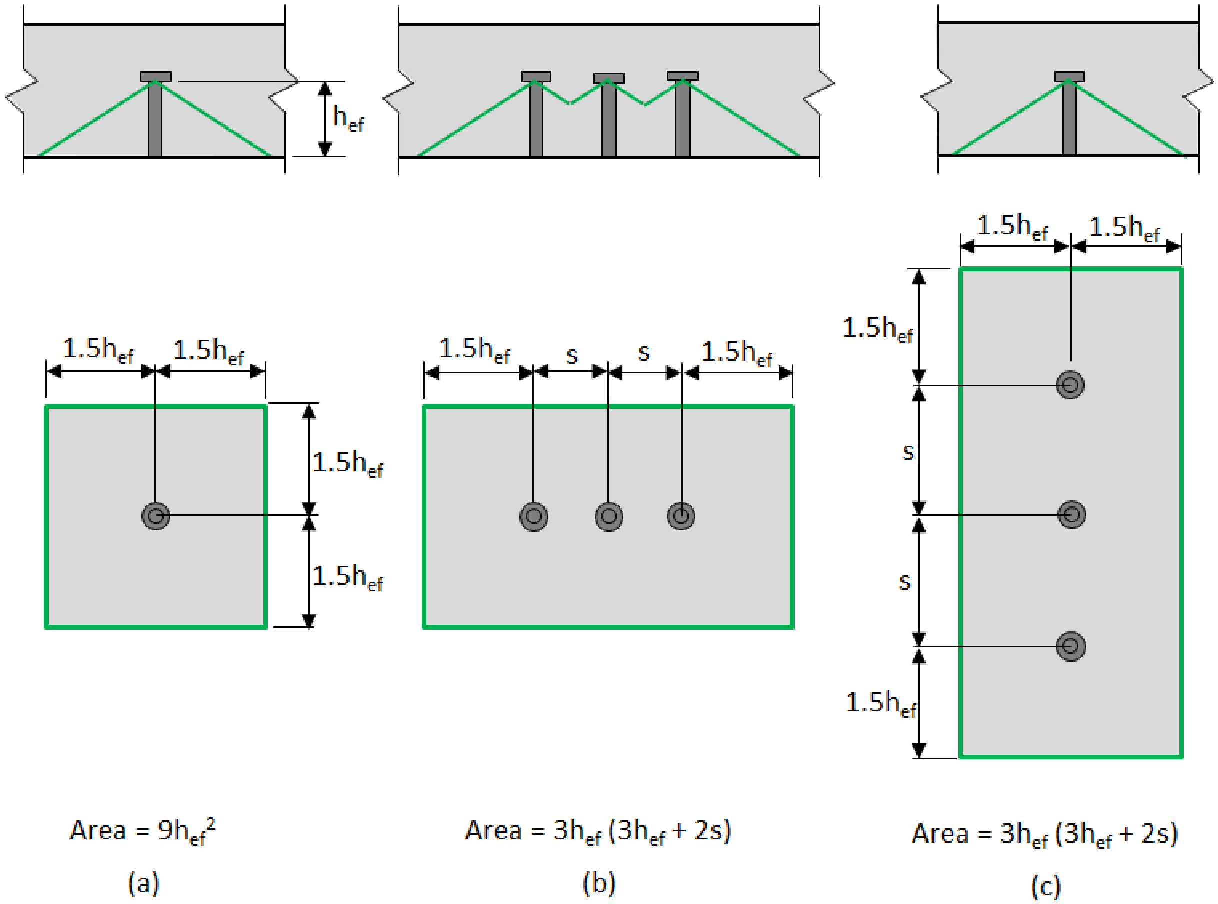


Figure C.2-4: Dimensioned Projected Concrete Cone Failure Areas for (a) 1 Stud (b) 3 studs Spaced Transversely (c) 3 Studs Spaced Longitudinally, all without a Haunch

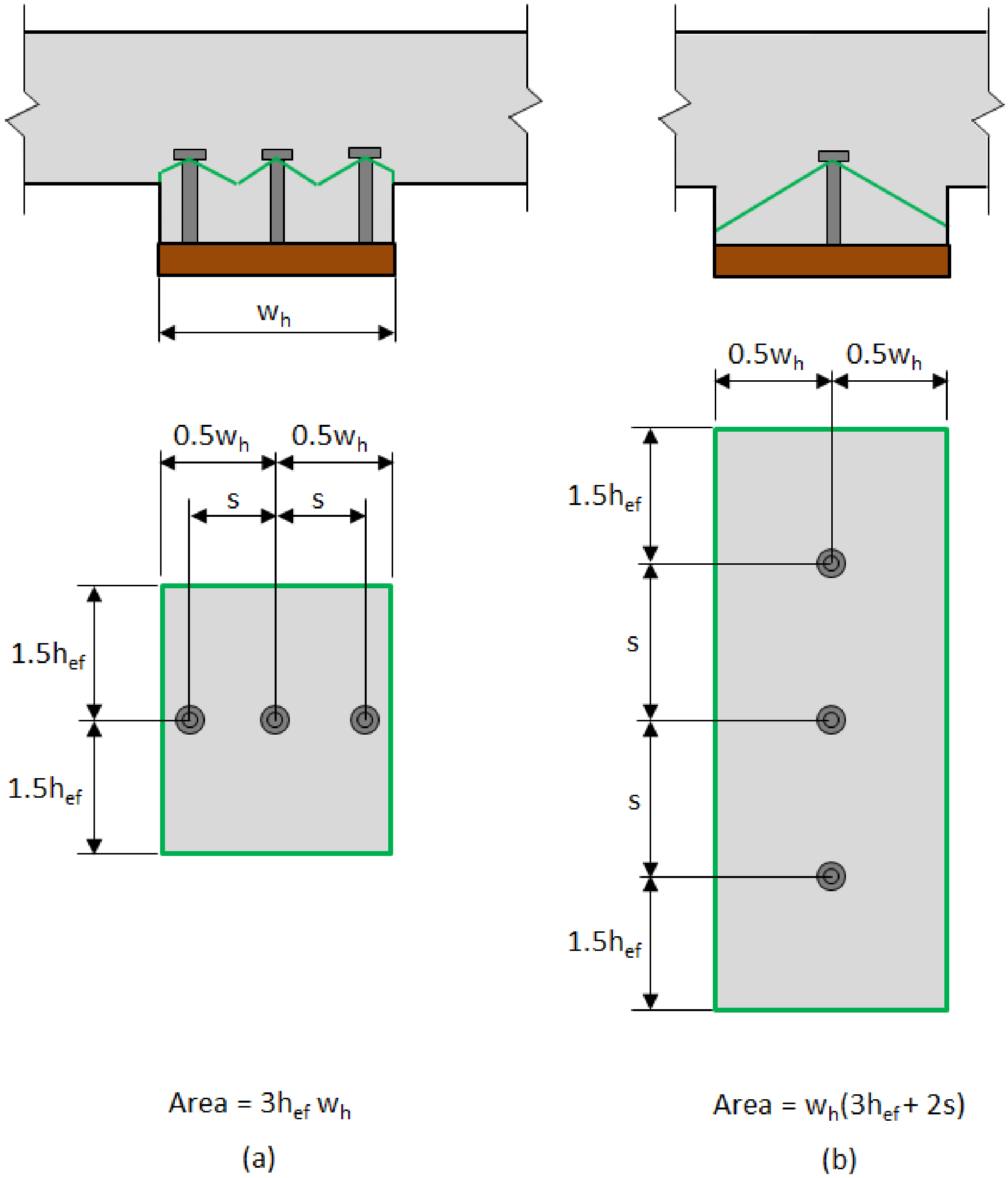


Figure C.2-5: Dimensioned Projected Concrete Cone Failure Areas for (a) 3 Studs spaced Transversely and (b) 3 Studs Spaced Longitudinally, both in a Haunch

The shear studs connecting the deck to the fractured girder must have sufficient tension capacity to develop the plastic beam mechanism in the bridge deck. The required shear stud tensile capacity is estimated by using the model of the bridge deck shown in Figure C.2-6 below.

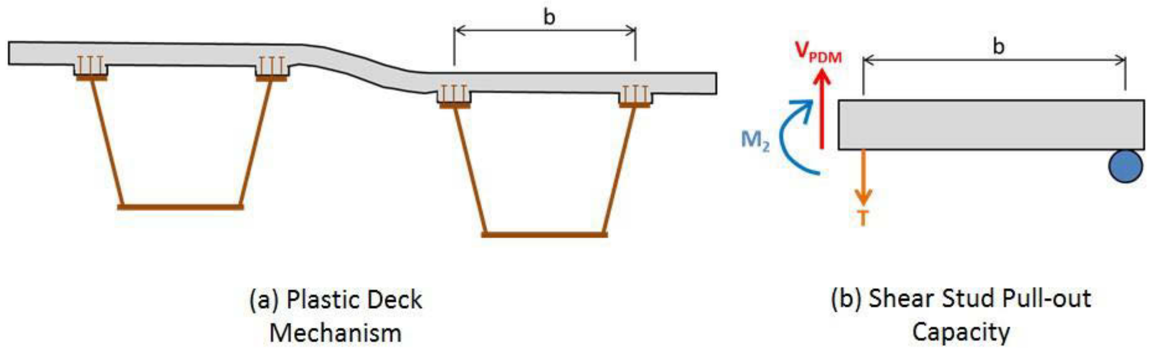


Figure C.2-6: (a) Plastic Deck Mechanism and (b) Shear Stud Pull-out Capacity

The required tension capacity of the group of shear studs included in the strip can be calculated as:

$$T \geq \frac{M_2}{b} + V_{PDM}$$

where:

T = Required tensile capacity of the shear stud group in a strip (k).

M_2 = Positive moment capacity of the deck strip at the top flange of the fractured girder (k-ft).

b = Distance between the mid-width of the top flanges of the fractured girder (ft).

V_{PDM} = Shear from the plastic deck mechanism (k).

If the tensile capacity of the shear studs is exceeded, the flange of the fractured girder will pull out of the bridge deck and the beam mechanism in the deck between the girders will not form.

Step 8:

Check the shear capacity of the composite section at the supports due to torsion and bending for Extreme Event III, V_{EEIII}.

The entire weight of the bridge and live load are applied to the intact girder. Calculate the shear, which is developed at the end of the span due to this loading, as:

$$V_{EEIII} = 1.2(1.10V_{DL} + 1.10V_{LL})$$

where:

V_{DL} = Shear due to dead load.

V_{LL} = Shear due to live load.

The unsupported load, which is first carried by the fractured girder, now has to be transferred to the intact girder. The eccentricity between the chord of the intact girder bearings and the center of gravity (CG) leads to a torque that is applied to the intact girder in addition to all the transferred loads. Results from the experimental testing program showed that the torsion introduced from the fractured girder into the intact girder was nearly symmetrical, indicating that the torque was resisted equally at each end of the intact girder (Barnard, et.al. 2010). Therefore, for simplicity, assume that the intact girder had symmetrical torsional boundary conditions so that each end resists one-half of the total applied torque.

Based on the assumption of symmetrical torsional boundary conditions, the torque of the dead load under Extreme Event III can be computed as:

$$T_{DL,EEIII} = (1.2)(1.1)\sum W_{DL} e_{DL}$$

where:

$T_{DL,EEIII}$ = Torque due to dead load under Extreme Event III (k-ft).

W_{DL} = Dead load of steel girders, deck, rails, etc. (k).

e_{DL} = Distance from each dead load component to centerline of support of intact girder for straight girders plus effect of curvature for curved girders (ft).

Based on the assumption of symmetrical torsional boundary conditions, the torque of the live load for Extreme Event III can be computed as:

$$T_{LL,EEIII} = (1.2)(1.1) W_{LL} e_{TL}$$

where:

$T_{LL,EEIII}$ = Torque for live load under Extreme Event III (k-ft).

W_{LL} = Live load (k).

e_{TL} = Distance between intact girder's centerline and truck (assuming lane load is coincident with the truck) plus effect of curvature for curved girders (ft).

When the girders have a horizontal radius of curvature, calculate the eccentricity as the distance between the center of gravity of the loads and the line of support for the intact girder. The center of gravity for nonprismatic curved girders can be determined using the equations in Barnard, et.al. 2010.

Assuming that one-half of the calculated torque is applied to each end of the intact girder, the shear flow, q, of the closed section can be calculated as:

$$q = \frac{1}{2}\frac{T_{DL,EE\ III} + T_{LL,EE\ III}}{2A}$$

where:

A = Area enclosed by the mid-thickness of the composite box girder section (in^2).

Check the concrete deck to ensure that it has adequate capacity to resist the shear force due to torsion. According to <Equation C5.7.3.3-1>, calculate shear capacity of reinforced concrete, V_s, as:

$$V_s = \frac{A_t f_{yt} b}{s}\cot(\theta)$$

where:

b = Width of the concrete deck between the top flanges (in).

A_t = Area of a reinforcement bar in the transverse direction (in^2).

s = Spacing between the reinforcement bars (in).

θ = Angle of shear crack with the horizontal plane (degrees).

Check that the shear capacity of reinforced concrete, V_s, is greater than the shear due to torsion calculated as:

$$V_{TORSION} = qb$$

Step 9:

Check the shear stress due to torsion for every component of the composite section.

The entire weight of the bridge and live load are applied to the intact girder.

I. Calculate the shear stress in the webs due to torsion as:

$$t_{TORSION\ WEB} = \frac{q}{t_{WEB}}$$

II. Calculate the shear stress in the webs due to bending as:

$$t_{FLEXURAL\ WEB} = \frac{V}{(2d_{WEB}t_{WEB}\cos(\beta))}$$

where:

d_{WEB} = Height of web (in).

β = Angle of web inclination (degree).

V = One-half of the total factored load on the span (k).

III. Calculate the shear buckling stress as:

$$t_n = C\ 0.058\ f_{yw}$$

where:

C = Ratio of shear-buckling resistance to the shear yield strength, <Article 6.10.9.3.2>.

IV. Ensure that the summation of the shear due to torsion ($t_{TORSION\ WEB}$) and bending ($t_{FLEXURAL\ WEB}$) is less than or equal to the shear-buckling stress.

$$t_n \geq t_{TORSION\ WEB} + t_{FLEXURAL\ WEB}$$

V. Check the bottom flange at the pier for combined shear and compression according to <Article 6.11.8.2.2.>.

VI. Check the end diaphragm and its connection to both girders to ensure that it has adequate resistance to the torque applied to the intact girder. This applied torque is resisted by a couple generated by the bearings of the two girders (bearing reactions). The reaction at the bearing of the fractured girder is equal to the torque applied to the intact girder divided by the distance between the bearings of the two girders. In the case of a continuous girder, the interior support is not as critical as the end support because some of the applied torque is resisted by the continuous girder. Thus, it is always critical to check the end diaphragm of the end support.

The forces acting on each side of the end diaphragm are calculated as follows:

$$V_{ED} = \frac{T_{DL,EE\ III} + T_{LL,EE\ III}}{l_b}$$

where:

V_{ED} = Forces acting on each side of the end diaphragm (k).

l_b = Distance between bearings of the two tub girders (ft).

Calculate the nominal shear resistance, V_n, of the end diaphragm according to <Article 6.10.9.2>.

Ensure: $V_{ED} < V_n$.

C.3 Redundancy Evaluation

Following the steps outlined in Section C.2, the redundancy level of a twin steel tub girder bridge can be evaluated. If the bridge under investigation satisfies the following conditions, the bridge has sufficient strength to sustain load without collapsing:

- Intact girder has adequate shear and moment capacity.
- Deck has adequate shear capacity.
- Shear studs have adequate tension capacity.

If the bridge only satisfies the first two conditions, it may still sustain load without collapsing. Under these conditions, a refined analysis can be used to evaluate the ability of the deck to transmit load to the intact girder without the shear studs connecting the deck to the fractured girder.

C.4 References

Barnard, T, C.G. Hovell, J.P. Sutton, J.S. Mouras, B.J. Neumann, V.A. Samaras, J. Kim, E.B. Williamson, and K.H. Frank. "Modeling the Response of Fracture Critical Steel Box-Girder Bridges*". FHWA/TX-10/9-5498-1. Center for Transportation Research, The University of Texas at Austin, Austin, TX., (https://ctr.utexas.edu/wp-content/uploads/pubs/9_5498_1.pdf), 2010.

Mouras, J.M., J.P. Sutton, K.H. Frank, and E.B. Williamson, "The Tensile Capacity of Welded Shear Studs", FHWA/TX-09/9-5498-2. Center for Transportation Research, The University of Texas at Austin, Austin, TX., (https://ctr.utexas.edu/wp-content/uploads/pubs/9_5498_2.pdf), 2008.

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Stith, J., A. Schuh, J. Farris, B. Petruzzi, T. Helwig, E. Williamson, K. Frank, M. Engelhardt, and H. J. Kim, "*Guidance for Erection and Construction of Curved I-Girder Bridges*", Technical Report FHWA/TX-10/0-5574-1. Center for Transportation Research, The University of Texas at Austin, Austin, TX., (https://library.ctr.utexas.edu/ctr-publications/0-5574-1.pdf), 2010.

Appendix D Lean-On Bracing Design Guide

D.1 Overview

Lean-on bracing is a method of preventing lateral torsional buckling in bridge girders and is primarily designed to withstand the forces that are expected to occur during the construction of the bridge deck. In steel I-girder bridges, conventional detailing practice is to provide braces between adjacent girders across the full width of the bridge, as shown in
Figure D.1-1. Cross-frames often represent one of the most expensive components of a steel bridge, per unit weight, due to the significant welding/bolting and handling requirements. Therefore, it is beneficial to refine the design and detailing of cross-frames.

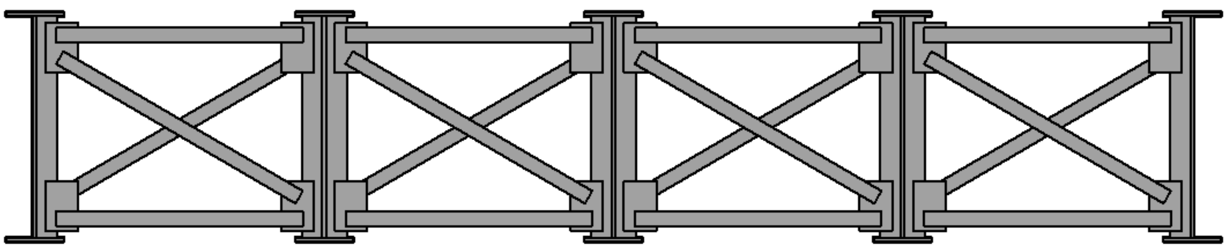


Figure D.1-1: Typical Cross-frame Bracing Line

Lean-on bracing consists of struts that transfer forces to strategically placed cross-frames within a given bracing line, as shown in Figure D.1-2. Cross-frames are positioned to distribute stiffness throughout the system and to help minimize the magnitudes of live load induced forces. Lean-on braces offer a cost-effective solution by combining a torsional bracing system (cross-frames) with lateral braces (struts). The method can be used on straight bridges with or without skew. Figure D.1-3 shows an in-service bridge with a lean-on bracing system.

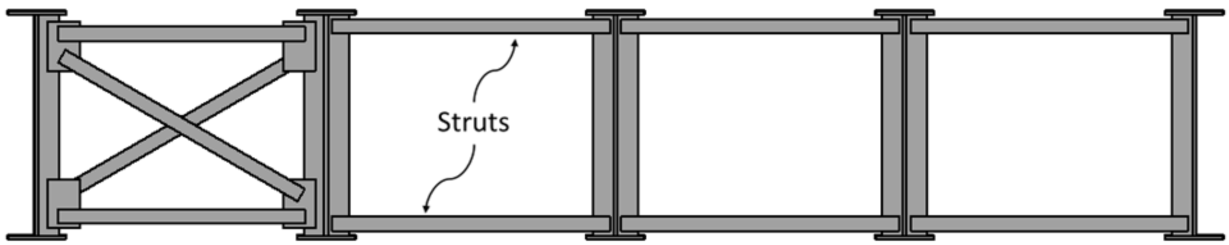


Figure D.1-2: Example Lean-on Bracing Line

If properly detailed and distributed along the length and width of the bridge, lean-on bracing systems can significantly reduce the number of full cross-frames required while potentially improving the long-term bridge behavior. The following are some benefits to using lean-on bracing concepts:

- Leads to fewer cross-frames on the bridge. This decreases fabrication and erection costs.
- Reduces fit-up issues during girder erection and construction.
- Can reduce construction time.
- Simplifies future inspections.
- Fewer fatigue prone details. Fatigue cracks are commonly found around locations of cross-frames and diaphragms during routine maintenance inspections. These cracks form due to large stress concentrations in the girder due to cross-frame and diaphragm forces induced by truck traffic on the bridge. This is particularly true for skewed bridges.

The TxDOT lean-on bracing design practice is based on the AASHTO LRFD Bridge Design Specifications, *Cross-Frame and Diaphragm Behavior for Steel Bridges with Skewed Supports (*0-1772, Helwig 2003) and *Refined Design Methods for Lean-On Bracing (*0-7093, Bjelland et. al, 2026).

The objective of 0-1772 was to improve the understanding of bracing behavior of cross-frames and diaphragms in steel bridges with skewed supports. General bracing requirements were developed, and new cross-frame and diaphragm details were proposed to minimize fatigue problems at bracing locations. The project included experimental and computational studies, as well as the development of a lean-on bracing design approach. Research project 0-1772 was utilized in the design of three skewed steel I-girder bridges in Lubbock, Texas. An associated implementation study was funded by TxDOT (5-1772) to monitor the performance of lean-on bracing in one of the Lubbock bridges during construction and under truck loading.

Research project 0-7093 was initiated to develop refined methods for designs utilizing lean-on bracing concepts, optimal layouts of cross-frame systems, and to provide recommendations for design parameters such as in-plane girder stiffness, effects of eccentric connections, and cross-frame type. The scope of 0-7093 included monitoring and field testing of bridges utilizing lean-on bracing, finite element modeling, parametric studies, and refinement of existing design expressions.

Figure D.1-3: Example of Bridge with Lean-on Bracing System

D.2 Lean-on Bracing Layouts

When using lean-on bracing concepts, the term “bracing layout” refers to the specific distribution of cross-frames and struts throughout a given bridge system. An example lean-on bracing layout is shown in Figure D.2-1.

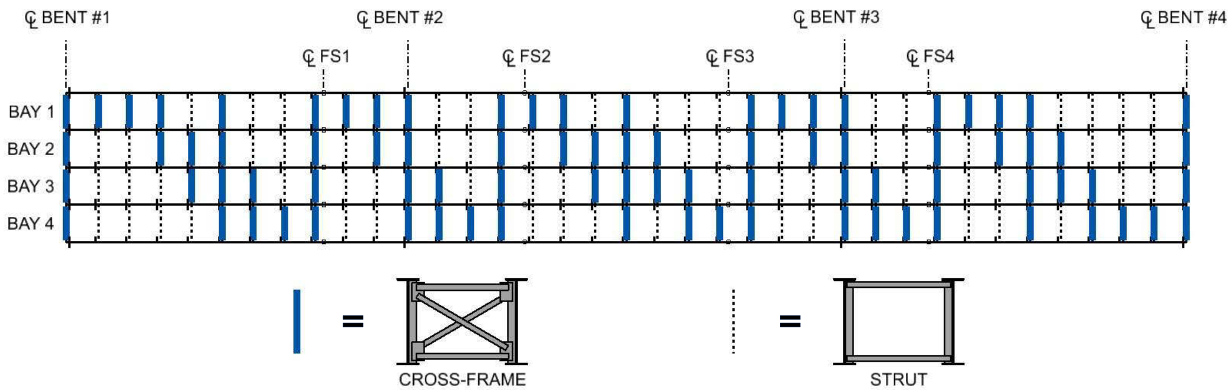


Figure D.2-1: Example Lean-on Bracing Layout

The total stiffness and buckling capacity of a given bridge system are directly tied to the bracing layout. Poorly distributed cross-frames can result in the significant degradation of several design parameters: including the brace stiffness of each cross-frame line, the in-plane girder stiffness of the system, and the system buckling capacity. Selection of an effective and efficient layout is, therefore, critical. Due to the substantial number of possible lean-on bracing layouts, the 0-7093 research team conducted a multi-step parametric study to identify the most effective layouts.

In general, it was found that lean-on bracing lines performed best when the distribution of cross-frames was balanced about the mid-point of a given span (both transversely and longitudinally). Additionally, an increased brace efficiency was observed when the cross-frame layout had both longitudinal and transverse continuity. This continuity is achieved by maintaining a continuous link between the full cross-frames within the system. In the longitudinal direction, this continuity is maintained by having cross-frames of adjacent bracing lines within the same bay. In the transverse direction, continuity comes from having adjacent cross-frames within the same bracing line. These findings led to the categorization of two primary layout effects:

1. Global layout effect
2. Local layout effect

The global layout effect is based on the distribution of the cross-frames within a span. This distribution should be reasonably balanced about the mid-point of a given span, as shown in Figure D.2-2, aiming for opposite sections of the span to have an equal number of cross-frames.

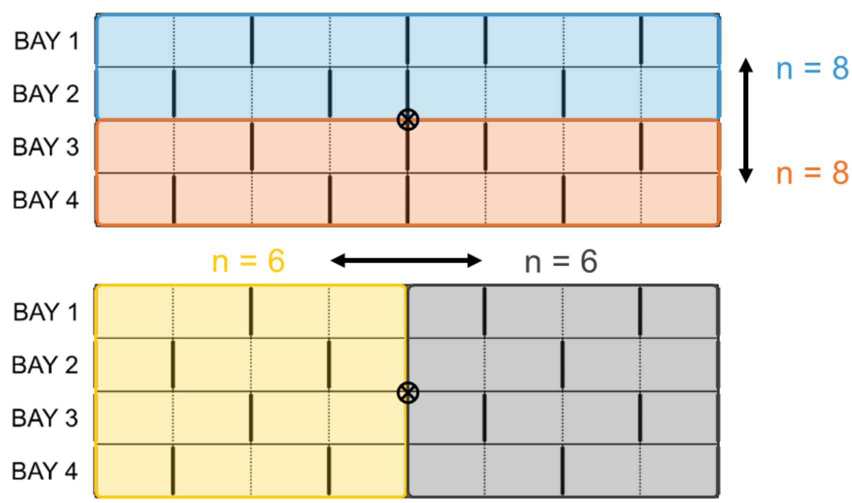


Figure D.2-2: Global Layout Effect – Balanced

Imbalances in the distribution, as illustrated in Figure D.2-3, lead to lower system stiffness and a reduction in the maximum moment capacity. However, a perfectly balanced system is not required.

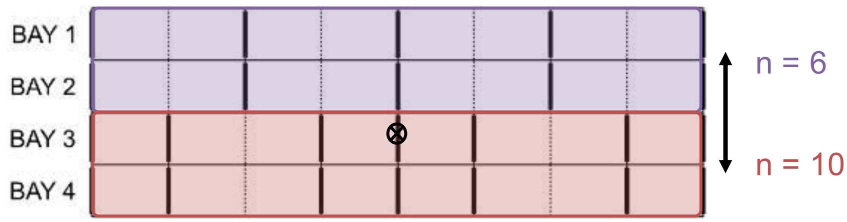


Figure D.2-3: Global Layout Effect – Unbalanced

Local layout effects are based on the interaction between adjacent bracing lines, which is highlighted in Figure D.2-4. A configuration that fails to link (or pair) cross-frames in consecutive bracing lines results in a lower maximum moment capacity. It also alters the displacement distribution, which can increase the susceptibility to amplification of geometric imperfections.

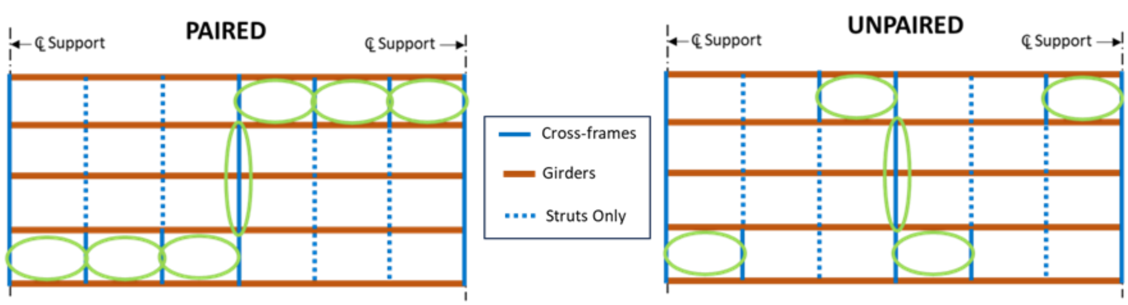


Figure D.2-4: Local Layout Effect

The combination of local and global layout effects was used to develop the layout criteria used in this guide. Research project 0-7093 considered various system geometries, and the report discusses the layout selection process in more detail.

D.2.1 Layout Selection

The development of the lean-on bracing design expressions, which are described in Section D.3, assumes that the system reasonably satisfies the following layout criteria:

- Systems must have a minimum of four girders.
- The number of torsional braces should be reasonably balanced transversely and longitudinally about the mid-point of each span.
- Each bracing line must have at least one torsional brace.
- Each bay along the length of the bridge should have approximately the same number of torsional braces.
- The number of grouped leaning girders must not exceed three in a given bracing line.
- A full line of torsional braces must be provided along each support line and at (or near) midspan.
- The first/last intermediate torsional braces near skewed supports should be placed in accordance with <Article C6.7.4.2.1>.

Providing permanent or temporary torsional bracing in the vicinity of field spliced can have a beneficial effect on girder stability during erection. Using these rules, four lean-on bracing layouts have been outlined for designers to use. These layouts and their appropriate application are shown in Table D.2.1-1 and Table D.2.1-2.

D.2.2 Layout Exception

Overall, the layouts shown in Table D.2.1-1 and Table D.2.1-2 follow the layout criteria with one exception; the checkerboard layout does not fully link adjacent cross-frame lines. As a result, excessively wide systems using the checkerboard pattern may lose some efficiency; and is, therefore, not recommended for systems exceeding six girders.

Table D.2.1-1: Lean-on Layouts for Bridge Systems with Support Skew ≤20°

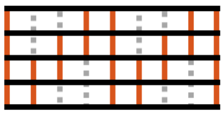
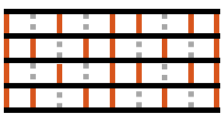
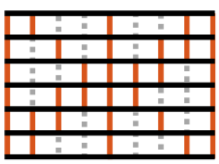
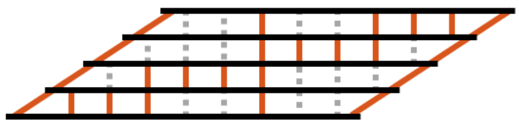
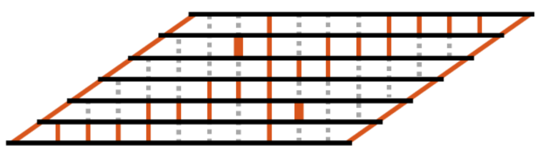
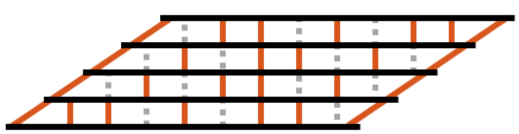
Layout	# of Girders	Notes
Zig-Zag 	4 - 6	Can be difficult to achieve this pattern in shorter spans
Checkerboard 	4 - 6	Good for any span length
"X" 	≥ 6	Layout for wide systems

Table D.2.1-2: Lean-on Layouts for Bridge Systems with Support Skew >20°

Layout	# of Girders	Notes
Diagonal  	≥ 4	Preferred layout for skewed systems. Wide systems may need additional cross-frames installed to reduce groups of leaning girders and aid in stability during erection.
Checkerboard 	4 to 6	Good for any span length

D.3 Lean-on Bracing Design Methodology

D.3.1 Introduction

This section outlines the TxDOT process for the design of lean-on bracing systems. The focus of this method is on stability design requirements and is, therefore, intended to parallel <Article 6.7.4.2.2> and <Article 6.10.3.4.2>.

In general terms, the following list provides an overview of the design procedure elements:

- Select an appropriate layout.
- Check buckling capacities.
- Satisfy stiffness requirements.
- Satisfy strength requirements.
- Perform other necessary AASHTO checks.

D.3.2 Lean-on Bracing Design

Lean-on bracing design begins with selecting an appropriate cross-frame arrangement for the structure based on the rules discussed in Section D.2.

Compare the buckling capacity of the system to the expected factored construction loads. If the system does not have sufficient buckling capacity, revise the superstructure before sizing cross-frame members.

Size cross-frame members based on the area needed to meet stiffness and strength requirements. Investigate each brace line to determine the true minimum member size because the required stiffness, provided stiffness, and force distribution all vary depending on brace-line configuration.

The design must also consider all other applicable AASHTO design requirements such as connection design, fatigue, etc.

D.3.2.1 Effective Girder Properties

This guide assumes that the girders have already been designed according to this manual. For girders with flange transitions (non-prismatic sections), effective girder dimensions may be used in the design of the bracing system. An approach for calculating effective element dimensions can be found in <Article D6.6.3> and is represented by the equation below (Reichenbach et al. 2020). The effective flange width and web thickness can be calculated using the same equation by substituting flange thickness (t_f) values for flange width (b_f) and web thickness (t_w) values.

Equation D.1 <Equation D6.6.3-5>

$$t_{f\,eff} = t_{f\,small}[1-(1-x_{small})^2] + t_{f\,2}(1-x_{small})^2$$

where:

$t_{f\,eff}$ = Effective flange thickness (in).

$t_{f\,small}$ = Smallest flange thickness (in).

x_{small} = Fraction of the length with smallest flange thickness over the span length.

$t_{f\,2}$ = Second smallest flange thickness (in).

D.3.2.2 Determine cross-frame locations

To ensure the optimal distribution of cross-frames within a bridge framing system, research project 0-7093 identified four lean-on bracing layouts as the most effective. These layouts and their appropriate application are described in Section D.2 and are shown in Table D.2.1-1 and Table D.2.1-2. The following design expressions were developed assuming that the layout selection criteria of Section D.2 are used.

D.3.2.3 Lateral-Torsional Buckling Capacities

A primary function of a bridge girder bracing system is to ensure that the girders have adequate capacity to withstand both conventional lateral-torsional buckling, as shown in Figure D3.2.2-1 (A), and global buckling as shown in Figure D.3.2.2-1 (B).

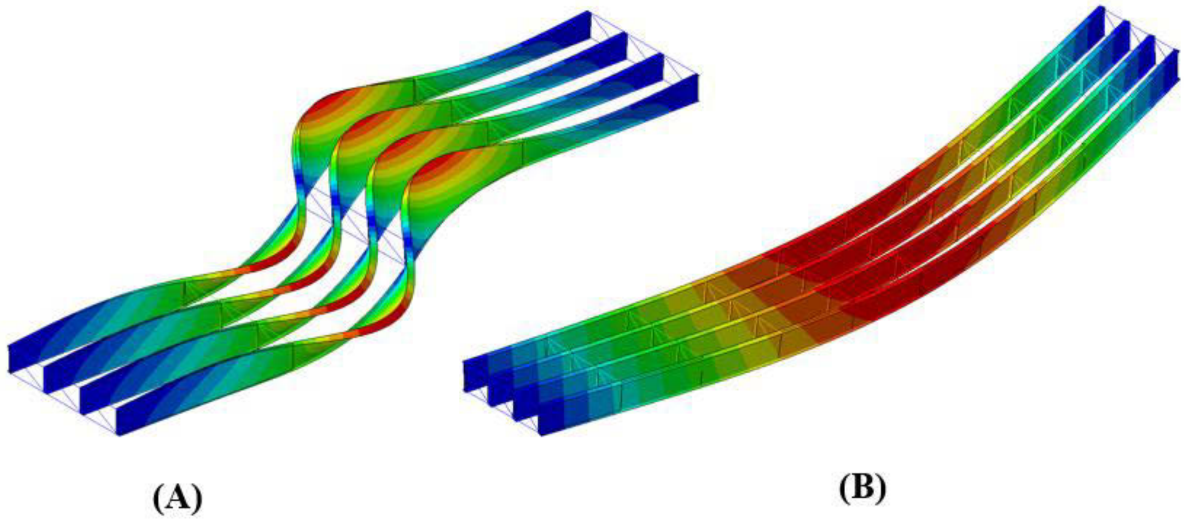


Figure D.3.2.3-1: Conventional (A) and Global (B) Lateral-Torsional Buckling Modes

Typically, the critical stage for checking these failure modes is during concrete deck placement, when the non-composite steel section is required to carry all construction loading. The following sections describe the process of verifying that the bridge has sufficient buckling capacity during construction. The maximum applied factored moments from construction loads must be compared to both the system global buckling and conventional lateral-torsional buckling capacities.

D.3.2.3.1 Calculate Moment Demand

Calculate the factored moment loading at each brace line (M_u). This deviates from <Article 6.7.4.2.2> because the variable brace stiffness within a lean-on system is best assessed on a per-brace line basis. The applied loading should be factored in accordance with <Article 3.4.2>. For systems where the cross-frame members are considered as Secondary Members, a load combination of 1.25 x (DC) + 1.50 x (construction live load) may be used. Otherwise, use 1.40 x (DC + construction live load).

The global buckling limit ($0.7M_{gs}$) should be compared to the absolute value of the largest moment ($+$ or $-$) within the span being investigated, in accordance with <Article 6.10.3.4.2>. This moment should not be taken at the support because it is often too conservative to do so and may result in unnecessarily large brace members.

D.3.2.3.2 Global Buckling Capacity

The global buckling capacity (M_{gs}) predicts the maximum moment each girder can withstand before the system buckles globally. The expression below was initially derived for conventional bracing layouts (Fish et al. 2025) but was adjusted for lean-on bracing systems by way of a "layout factor," $\mathrm{C_{LO}}$ (Bjelland et al. 2026). This expression assumes that the bracing between girders is stiff enough for the system to behave as a unit. If the global buckling capacity is insufficient, adding warping restraint to the system increases the capacity. One way this can be done is by specifying staged deck placement at the ends of the span and allowing the deck to harden before pouring the rest of the unit (Bjelland et al. 2026). It is important to note that the global buckling capacity is not sensitive to the number of intermediate brace lines within the span, so adding brace lines will not significantly increase the system capacity.

Equation D.2 (Bjelland et al., 2026)

$$M_{gs} = C_{LO} C_{bs} \frac{\pi^2 SE}{(KL)^2} \sqrt{I_{y\,eff} I_x \alpha_x}$$

where:

M_{gs} = global buckling capacity (k-in).

C_{LO} = 0.95 for bridges with support skew less than or equal to 20 degrees (including zero skew) and 0.85 for bridges with skewed supports greater than 20 degrees.

C_{bs} = 1.1 for simple spans or partially erected continuous spans, 2.0 for fully erected continuous spans.

S = Girder spacing (in).

E = Modulus of elasticity of the girders (ksi).

K = 1.0 for bridges with support skew less than or equal to 20 degrees (including zero skew) and 0.8 for bridges with skewed supports greater than 20 degrees.

L = Span length (in).

$I_{y\,eff}$ = Effective moment of inertia of the girder about the y-axis (in^4).

Equation D.3 (Yura et al., 2001)

$$I_{y\,eff} = I_{yc} + (t/c)I_{yt}$$

where:

t, c = Distances from the geometric centroid of the section to the tension and compression flange centroids, respectively (in)

I_{yc}, I_{yt} = Moment of inertia of the compression and tension flanges about an axis in the plane of the web, respectively (in^4)

I_x = Moment of inertia of the girder about the x-axis (in^4)

α_x = System warping stiffness factor

Equation D.4 (Fish et al., 2025)

$$\alpha_x = \frac{\left(n_g^2 - 1\right)}{12}$$

n_g = Number of girders

Verify that the maximum factored moment loading is less than seventy percent of the calculated global buckling capacity ($M_u < 0.7M_{gs}$). Although the global buckling moment capacity equation agrees well with eigenvalue buckling analyses, the limit of $0.7M_{gs}$ is required by <Article 6.10.3.4.2> to avoid second-order amplification of deformations (Han and Helwig 2020).

D.3.2.3.3 Conventional Lateral-Torsional Buckling Capacity

Equation D.5 <Equation D6.6.3-9>

$$M_{cr} = F_{e\,eff} S_{xc\,eff} = \frac{S_{xc\,eff} C_b \pi^2 E}{\left(\frac{L_b}{r_{t\,eff}}\right)^2} \sqrt{1 + 0.078 \frac{J_{eff}}{S_{xc\,eff} h_0} \left(\frac{L_b}{r_{t\,eff}}\right)^2}$$

where:

M_{cr} = Conventional lateral-torsional buckling capacity (kip-in).

$F_{e\ eff}$ = Elastic LTB stress for the prismatic unbraced length (ksi).

$S_{xc\,eff}$ = Effective elastic section modulus about the major axis of the section (in^3).

C_b = Moment-gradient factor.

L_b = Unbraced length of the girder (in).

J_{eff} = Torsional constant of the effective section (in^4).

h_0 = Distance between flange centroids of the effective section (in).

$r_{t\,eff}$ = Radius of gyration of the effective section calculated in the following equation (in).

Equation D.6 <Equation D6.6.3-7>

$$r_{t\,eff} = \frac{b_{fc\,eff}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_{c\,eff}}{b_{fc\,eff}}\frac{t_{w\,eff}}{t_{fc\,eff}}\right)}}$$

where:

$b_{fc\,eff}$ = Effective flange width corresponding to the flange subject to flexural compression within the unbraced length under consideration (in).

$D_{c\,eff}$ = Depth of the web in compression in the elastic range for the effective cross-section within the unbraced length under consideration (in).

$t_{fc\,eff}$ = Effective flange thickness corresponding to the flange subject to flexural compression within the unbraced length under consideration (in).

$t_{w\,eff}$ = Effective web thickness within the unbraced length under consideration (in).

Verify that the factored moment loading at each cross-frame line is less than the calculated conventional lateral-torsional buckling capacity ($M_u < 1.0M_{cr}$).

D.3.2.4 Minimum Required Brace Member Area

Now that the moment demand is known and the girders meet the global and lateral-torsional buckling criteria, the bracing system can be designed to satisfy the stiffness requirements. The system stiffness provided must be equal to, or larger than, the required system stiffness.

Equation D.7 <Equation 6.7.4.2.2-1>

$$(\beta_T)_{prov} \geq (\beta_T)_{req}$$

As will be shown in subsequent sections, this relationship can be used to determine the required sizes of the brace members used in the cross-frames.

D.3.2.4.1 Provided System Stiffness

The provided stiffness of the torsional bracing system is a function of the in-plane girder, cross-frame (or diaphragm), and cross-section stiffnesses. These individual components are combined by the classic equation for springs in series. Therefore, the provided stiffness at each brace line can be predicted using the following expression (Yura, 1992):

Equation D.8 <Equation 6.7.4.2.2-6>

$$(\beta_T)_{prov} = \left(\frac{1}{\beta_g} + \frac{1}{\beta_{br}} + \frac{1}{\beta_{sec}}\right)^{-1}$$

where:

$(\beta_T)_{prov}$ = Total provided torsional bracing system stiffness (kip-in/rad).

β_g = In-plane girder stiffness (kip-in/rad).

β_{br} = Cross-frame sStiffness of a given bracing line (kip-in/rad).

β_{sec} = Cross-sectional stiffness (kip-in/rad).

The equation indicates that $(\beta_T)_{prov}$ will always be less than the smallest of the three individual stiffness components. Therefore, the overall stiffness of a torsional brace is effectively limited by the most flexible component in the system.

D.3.2.4.2 Required System Stiffness

According to <Article 6.7.4.2.2>, when using braces that are at least eighty percent of the depth of the girder depth, the following expression for the required brace stiffness can be used.

The expression for determining the required system stiffness is shown below:

Equation D.9 <Equation 6.7.4.2.2-2>

$$(\beta_T)_{req} = \frac{2.4L}{\phi_{sb} n_{CFL} E I_{y\,eff}} \left(\frac{M_u}{C_b}\right)^2$$

where:

$(\beta_T)_{req}$ = required system stiffness (kip-in/rad).

M_u = Factored moment demand at the cross-frame line being investigated (kip-in).

ϕ_{sb} = LRFD reduction factor for stability bracing (0.8).

C_b = Moment gradient factor.

n_{CFL} = Number of bracing lines in the span.

$I_{y\,eff}$ = Effective moment of inertia about the y-axis (in^4).

D.3.2.4.3 In-plane Girder Stiffness

The in-plane (i.e., vertical) flexural stiffness of the bridge girders themselves contribute to the overall stiffness of the torsional bracing system. As shown in Figure D3.2.4.3-1, when the girders are subjected to a twist, the internal moment in the cross-frame is equilibrated by vertical shear forces acting at the ends of the brace. The vertical shear forces on the adjacent girders cause one girder to deflect upwards and the other to deflect downwards leading to a rigid body rotation. These deformations reduce the effectiveness of the brace. With a wider system, this displacement is reduced, as demonstrated by the four-girder system shown in the figure.

When calculating the provided system stiffness of a brace line at a support, β_g should be taken as infinity.

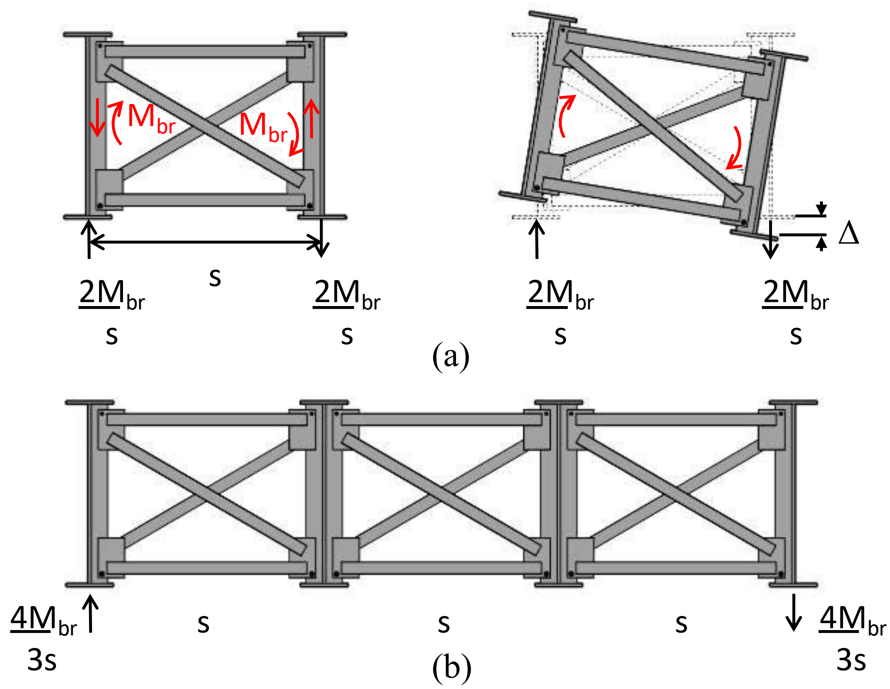


Figure D3.2.4.3-1: In-Plane Girder Stiffness

Calculate the in-plane girder stiffness by the equation below. As with global buckling capacity, if the in-plane girder stiffness is not sufficient, adding warping restraint to the system increases the stiffness. Adding warping restraint increases the global buckling capacity, and the in-plane girder stiffness because they are related. Staged deck placement at the ends of the span (Bjelland et al. 2026) can be used in this situation as well.

Equation D.10 (Bjelland et al., 2026)

$$\beta_g = C_{LO}^2 \frac{\pi^4 E I_x S^2 \alpha_x}{(KL)^4} L_{b\ avg}$$

where:

$L_{b\ avg}$ = The average unbraced length within the span under consideration (in)

This expression is a slight variation of the one presented by Bjelland et al. (2026), though it is functionally identical. Fish et al. (2025) showed that the appropriate discretization of the uniformly distributed in-plane girder stiffness ($\bar{\beta}_g$) is $L/(n_{CFL}+1)$. This assumed that all cross-frames are equally spaced. When this is this case, $L/(n_{CFL}+1) = L_b$. For the above expression, $L_{b\ avg}$ is the average unbraced length within the span.

D.3.2.4.4 Cross-Sectional Stiffness

The stiffness of the girder cross-section (β_{sec}) can have a significant effect on the total system stiffness. Insufficient cross-sectional stiffness leads to distortion of the web which reduces the effectiveness of the torsional braces. However, the cross-frames can prevent the web from distorting within the depth of the cross-frame. According to <Article 6.7.4.2.2>, when the braces are at least eighty percent of the web depth, the cross-sectional distortion stiffness can be taken as infinity. In typical TxDOT designs, the braces should meet this requirement. If not, guidance on calculating β_{sec} can be found in <Article 6.7.4.2.2> and in the 0-7093 research report.

D.3.2.4.5 Required Brace Stiffness

Since the provided system stiffness must be equal to or greater than the required system stiffness, the total required stiffness of the system can be related to the minimum required brace stiffness ($(\beta_T)_{req}$) by the following expression.

Equation D.11

$$(\beta_T)_{req} = \left(\frac{1}{\beta_g} + \frac{1}{\beta_{br\ req}} + \frac{1}{\beta_{sec}}\right)^{-1}$$

A simple rearrangement of this equation isolates the required brace stiffness term, as seen below. This is the brace stiffness needed at the cross-frame line that is associated with the required system stiffness ($\beta_{br\ req}$) being used in the expression.

Equation D.12 (Bjelland et al., 2026)

$$\beta_{br\ req} = \left(\frac{1}{(\beta_T)_{req}} - \frac{1}{\beta_g} - \frac{1}{\beta_{sec}}\right)^{-1}$$

It is worth noting that if the in-plane girder stiffness component is less than the required system stiffness, this expression results in a negative value. If this is the case, no amount of brace stiffness will be sufficient. The system must either be redesigned or warping restraint, such as staged deck placement, must be added (Bjelland et al. 2026).

The following expression is used to calculate the minimum brace member area ($A_{b\ req}$(in^2)) for each cross-frame line. This expression assumes that the same member is used for both the diagonals and the struts.

Equation D.13 (Bjelland et al., 2026)

$$A_{b\ req} = \beta_{br\ req} \frac{C_1 L_d^3 + (C_{strut} - C_2) S^3}{E S^2 h_b^2 R}$$

where:

R= Factor accounting for connection eccentricity (0.65 – during construction when girders are non-composite <Article 4.6.3.3.4c>).

L_d= Length of a diagonal member (in).

h_b= Height of the brace (in).

C_{strut}= Strut efficiency term related to the number of girders/braces and leaning girders.

C_1= Diagonal member term dependent on the bracing type.

C_2= Strut member term dependent on the bracing type.

To calculate the required brace member area, the following parameters must be identified (use Table D.3.2.4.5-1 if the layout is checkerboard):

n_c= Number of cross-frames in the bracing line.

n_g= Number of girders in the cross-section.

n_{lean}= Maximum number of grouped leaning girders in the bracing line as illustrated in Figure D.3.2.4.5-1.

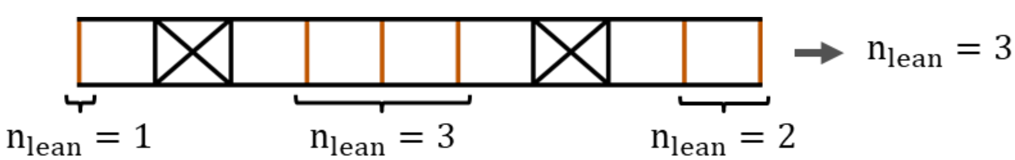


Figure D.3.2.4.5-1: Example of Identifying n_{lean}

Calculate the C_{strut} term, which is dependent upon the brace distribution within a given brace line as well as the overall layout configuration selected.

- For brace lines with no lean-on struts:

  Equation D.14 (Bjelland et al., 2026)

  $$C_{strut} = \frac{n_g + 1.75}{2n_g - 0.25}$$

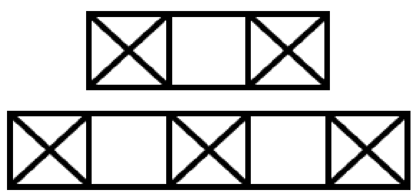
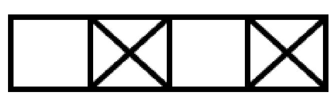
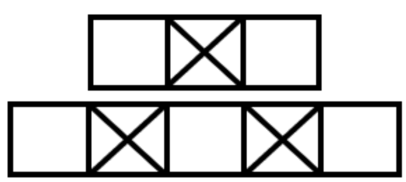
- For lean-on bracing lines using "Zig-zag", "X", or "Diagonal" layout configurations:

  Equation D.15 (Bjelland et al., 2026)

  $$C_{strut} = (0.70 - 0.05n_c)(n_{lean} + 1)^2$$

- If the checkerboard layout is selected, use the values from the following table.

Table D.3.2.4.5-1: Checkerboard Reference

Exterior Bay Bracing Condition	Example Bracing Line Configuration	n_{lean}	C_{strut}
Both exterior bays have a cross-frame		0	1.0
One exterior bay has a cross-frame		1	2.4
Neither exterior bay has a cross-frame		1	2.6

Calculate C_1 and C_2 using the appropriate expressions in the following table, for each brace line.

Table D.3.2.4.5-2: Constants for brace stiffness expression

Brace Type	C_1	C_2
Z	$\frac{n_g}{n_c}$	0
X	$\frac{n_g}{2n_c}$	$\frac{n_g}{2n_c}$
K	$\frac{2n_g}{n_c}$	$\frac{n_g}{4n_c}$

Calculate the required brace member area for each brace line and select members that satisfy the requirement. This completes the sizing of the brace member for adequate stiffness; however, the brace also needs to be checked for adequate strength. Strength checks are presented in the next section.

D.3.2.5 Verify Selected Brace Members Meet Strength Requirements

Initial imperfections in the girder and the stiffness of the individual braces create additional moments in the cross-frames. These moments cause forces to develop in both the struts and diagonals. The selected brace member should be checked for its capacity to withstand these forces.

D.3.2.5.1 Determine the Forces Acting on the Braces

Calculate the stability induced moment in the brace, $\mathrm{M_{br}}$ (kip-in), using the equation below (assuming the cross-frames are at least eighty percent of the girder depth.

Equation D.16 <Equation 6.7.4.2.2-14>

$$M_{br} = \frac{2.4L}{n_{CFL}EI_{y\,eff}}\left(\frac{M_u}{C_b}\right)^2\left(\frac{L_b}{500h_0}\right)$$

Calculate the stability induced brace force, F (kips), using the equation below.

Equation D.17 (Bjelland et al., 2026)

$$F = \frac{M_{br}}{h_b}$$

The brace member force calculated by Equation D.17 is the force induced as the braces stabilize the girders and do not include other construction force effects such as overhang brackets and wind (F_{con} (kips)). These additional forces should be added to the stability induced forces when checking the capacity of each member.

Equation D.18

$$F_{tot} = F + F_{con}$$

The force in the struts and diagonals of the cross-frame is dependent on n_c and n_{lean}, so it will vary depending on the cross-frame pattern in each bracing line. Calculate the maximum forces in the cross-frame diagonals, F_d (kips), and struts, F_s (kips), using the below equations.

Table D.3.2.5.1-1: Total Brace Member Forces for Lean-on Braces

Brace Type	$F_{d,tot}$	$F_{s,tot}$
Z or K	$\frac{F_{tot} L_d n_g}{S n_c}$	$(n_{lean} + 1)F_{tot}$
X	$\frac{F_{tot} L_d n_g}{2 S n_c}$	$n_{lean} F_{tot}$

Table D.3.2.5.1-2: Total Brace Member Forces for Conventional Braces

Brace Type	$F_{d,tot}$	$F_{s,tot}$
Z or K	$\frac{F_{tot}L_d n_g}{S n_c}$	F_{tot}
X	$\frac{F_{tot}L_d n_g}{2S n_c}$	$\left(1-\frac{n_g}{2n_c}\right)F_{tot}$

D.3.2.5.2 Check the Tensile Strength Capacity

Gross Section Yield:

The yield capacity ($\phi_y P_{ny}$ (kips)) of the bracing members must be greater than the cross-frame diagonal ($F_{d,tot}$) and strut forces ($F_{s,tot}$). Calculate the yield capacity using the below equation:

Equation D.19 <Equation 6.8.2.1-1>

$$\phi_y P_{ny} = \phi_y F_y A_{b\ prov}$$

where:

ϕ_y = Resistance factor for yielding (0.95).

F_y = Yield strength of the brace members (ksi).

$A_{b\ prov}$ = Gross brace member area (in^2).

Net Section Fracture:

The fracture capacity ($\phi_u P_{nu}$ (kips)) of the bracing members must be greater than the cross-frame diagonal ($F_{d,tot}$) and strut ($F_{s,tot}$) forces. Calculate the fracture capacity using the following equation:

Equation D.20 <Equation 6.8.2.1-2>

$$\phi_u P_{nu} = \phi_u F_u A_n R_p U$$

where:

ϕ_u = Resistance factor for fracture (0.8).

F_u = Tensile strength of the brace members (ksi).

R_p = Reduction factor for holes.

U = Shear lag reduction factor.

A_n = Net area of the brace member (in^2).

D.3.2.5.3 Check the Compressive Strength Capacity

The compressive capacity of each bracing member must exceed the total cross-frame diagonal ($F_{d,tot}$) and strut ($F_{s,tot}$) forces. The effective slenderness ratio should be calculated per <Article 6.9.4.4>. The factored compressive resistance of a single angle should be calculated using the provisions in <Article 6.9.4.1>.

D.3.2.6 Check Other Design Requirements

Check other important design requirements. The system must be checked to ensure that it satisfies all other applicable design requirements including but not limited to connection designs, varied load conditions, construction phasing, etc.

D.4 References

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