Navigating Routine Steel Bridge Design

AASHTO LRFD Bridge Design Specifications, 10th Edition

Smarter. Stronger. Steel.

Revised March 2025

FOREWORD

The design of bridges requires the efforts of engineers educated and experienced in structural design, specifically the unique aspects of bridge design. However, bridge design need not be complicated or challenging, particularly for the more routine bridges which form a large part of the inventory of transportation structures in the United States. In particular, the design of "routine steel I-girder bridges," a workhorse structure type, can be relatively simple if the engineer knows where to focus their efforts and is provided with guidance on how to streamline the more predictable, repetitive aspects of the design effort.

To this end, the National Steel Bridge Alliance (NSBA), a division of the American Institute of Steel Construction (AISC) has developed this *Guide to Navigating Routine Steel Bridge Design*. The goal of this Guide is to help designers navigate the comprehensive design provisions of the AASHTO *LRFD Bridge Design Specifications* (AASHTO LRFD BDS), identifying just the provisions that are applicable to the design of routine steel I-girder bridges, explaining how to apply those provisions, recommending practices proven to lead to economical designs, and suggesting ways to streamline the design effort. The Guide was originally written to align with the 9th Edition of the AASHTO LRFD BDS, and now has been updated to align with the 10th Edition.

This Guide is meant to be used as an interactive reference, rather than as a textbook read from cover to cover. The general flow of design tasks is outlined, and at any point in the design process the reader can quickly jump to a detailed Discussion of any particular AASHTO LRFD BDS provision. Through this process, a bridge engineer will not only find answers to specific questions, but can also increase their familiarity with, and understanding of, the AASHTO LRFD BDS.

We would like to gratefully acknowledge the support of Christopher Garrell (NSBA), who steered the development of this Guide and its recent update. Credit is also due to Kaylene Callicoatt (HDR), who helped assemble and edit the original version of the Guide. Finally, we would like to thank the numerous professionals who provided invaluable peer review comments during the first writing of this Guide, including: Travis Butz (Burgess and Niple), Matt Farrar (Idaho DOT), Jamie Farris (Texas DOT), Karl Frank (Consultant), Christina Freeman (Florida DOT), Dennis Golabek (WSP), John Holt (Modjeski and Masters), Ted Kniazewycz (Tennessee DOT), Shane Kuhlman (New Mexico DOT), Ronnie Medlock (High Steel Structures), Adam Price (Tennessee DOT), Curtis Rockiki (Texas DOT), Kevin Sear (AECOM), Tony Shkurti (HNTB), Jason Stith (Michael Baker International), Greg Turco (Texas DOT), Jeffrey Vetter (Idaho DOT), Dayi Wang (Federal Highway Administration), Wagdy Wassef (WSP), and Jaclyn Whelan (AECOM).

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February 2025

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SCOPE OF THIS GUIDE

This NSBA Guide to Navigating Routine Steel Bridge Design (this Guide) primarily addresses the design of steel superstructures for "routine steel I-girder bridges." The intent is to illustrate which provisions of the AASHTO *LRFD Bridge Design Specifications* (AASHTO LRFD BDS) are applicable to the design of these types of structures, and perhaps more importantly, which provisions are not applicable or are perhaps only partially or conditionally applicable or beyond the scope of superstructure design. In doing so, it is hoped that this Guide will help engineers streamline the design process, avoid unnecessary or misguided effort, and simplify their approach to the necessary tasks associated with the design of a more routine steel I-girder bridge. The intended audience includes, but is not limited to, those designers who may be less experienced with steel bridge design. This Guide is intended to be used in combination with the AASHTO LRFD BDS and should not be used as a substitute for the AASHTO LRFD BDS itself.

The definition of a "routine steel I-girder bridge" is provided below and is intended to encompass a large family of straight steel I-girder bridges with little or no skew, "routine" span lengths, and commonly used framing layouts and details. To keep the Guide focused, the following items are specifically excluded from the scope of this Guide:

- Barrier rail design: Standard details, often mandated by the local Owner-agency, are generally used, so design guidance per se is not needed.
- Deck design: The design of concrete decks for steel I-girder bridges is often governed by Owner-agency policy manuals (e.g., standard designs, pre-calculated design tables, etc.), and so their design is not addressed herein.
- Substructure and foundation design: The wide variety of types, configurations, materials, and conditions associated with substructure and foundation design make it difficult to provide a succinct set of guidelines that would be broadly applicable over all parts of the U.S.
- Bearing design: The variety of types of bearings and the associated variety of local Owner-agency or regional bearing design preferences make it difficult to provide a succinct set of guidelines that would be broadly applicable over all parts of the U.S. For the purposes of this Guide, the use of "routine" bearings (as defined under "Definition of a 'Routine Steel I-Girder Bridge' below) is assumed.

Given these scope limitations, consideration of the Extreme Event limit state, outside of the identification of the seismic zone for design, is also omitted from this Guide.

If a given bridge somehow falls partially outside the limits of the definition of a "routine steel I-girder bridge" or outside the exclusions of this scope, this Guide may still provide value to designers; in such cases, senior bridge engineers with extensive experience in steel bridge design should be consulted when determining if and how to apply any of the recommendations provided herein.

Conversely, designers should be cognizant of local Owner-agency policies which may supersede the recommendations and information presented in this Guide. In such cases senior bridge engineers with extensive experience in steel bridge design should be consulted when determining how to apply this Guide in conjunction with Owner-agency policy.

Finally, while the scope of this Guide is limited to helping bridge designers understand and correctly apply specific provisions of the AASHTO LRFD BDS, it is worthwhile to briefly address the broader scope of bridge design.

- **Correctness of structural design** – It is of the utmost importance to correctly design a bridge with sufficient strength to carry the intended loads without suffering undue distress or structural failure. The safety of the general public who travel on and under bridges is the overriding concern of a bridge designer.
- **Correctness of geometric information** – It is also important that pertinent geometric information be correctly, accurately, and precisely calculated and presented on the plans. Errors in the calculation and presentation of geometric information can result in costly delays, rework, and claims.
- **Correctness of estimated quantities** – It is also important that required estimated quantities, particularly quantities which form the basis of payment for a bridge, be correctly, accurately, and precisely calculated and presented on the plans. Errors in the calculation and presentation of estimated quantities can result in costly delays, rework, and claims.

DETERMINATION DEFINITIONS

Each Article of the AASHTO LRFD BDS is assigned a Determination of Applicability to the design of routine steel I-girder bridges. These Determinations of Applicability do not relieve designers of their responsibility to read, understand, and correctly apply the provisions of the AASHTO LRFD BDS, but instead are intended to aid designers in navigating and understanding those provisions.

The various Determinations are defined as follows:

1. **Applicable:** The Article, in its entirety, is fully applicable to the design of routine steel I-girder bridges
2. **Partially Applicable:** Parts of the Article are applicable to the design of routine steel I-girder bridges, other parts are not applicable; see the Discussion for explanation
3. **Conditionally Applicable**: Some or all of the Article may be applicable to the design of routine steel I-girder bridges depending on the circumstances; see the Discussion for explanation
4. **Not Applicable:** None of the Article is applicable to the design of routine steel I-girder bridges
5. **Beyond Scope of Superstructure Design:** Some or all of the Article may be applicable to some aspect of the design of routine steel I-girder bridges, but is not applicable to superstructure design; see the Discussion for explanation

TERMINOLOGY

For the purposes of this Guide, the following terminology is defined to avoid confusion:

AASHTO LRFD BDS: The term "AASHTO LRFD BDS" refers to the AASHTO *LRFD Bridge Design Specifications*, 10th Edition, 2024.

Article: The term "Article" (capitalized) refers to a specific numbered Article in the AASHTO LRFD BDS.

Commentary: The term "Commentary" (capitalized) refers to a specific numbered commentary section related to an Article in the AASHTO LRFD BDS.

Determination: The term "Determination" (capitalized) refers to the determination of applicability of a given Article in the AASHTO LRFD BDS to the design of routine steel I-girder bridges.

Discussion: The term "Discussion" (capitalized) refers to the discussion explaining the rationale behind a specific Determination in this Guide and/or providing guidance on how to streamline the design tasks or actions associated with the Article referenced.

Guide: The term "Guide" (capitalized) refers to this NSBA Guide to Navigating Routine Steel Bridge Design.

DEFINITION OF A "ROUTINE STEEL I-GIRDER BRIDGE"

For the purposes of implementing the recommendations of this Guide, a "routine steel I-girder bridge" is defined as the superstructure of a bridge meeting the following characteristics:

- Straight (non-curved) steel I-girders (rolled steel beams or welded steel plate girders).
- Straight (non-curved) deck.
- Framing such that flange lateral bending can be neglected (except for the effects of deck overhang brackets during construction and wind), specifically:
    - Skew not more than 20 degrees, where skew is measured as the angular deviation of the orientation of the supports from perpendicular to the centerline of the bridge.
    - Parallel supports or supports which are within 10 degrees of being parallel.
    - Contiguous cross-frames or diaphragms.
    - Parallel girders.
- Constant deck width.
- Skew Index less than or equal to 0.30, where the Skew Index is as defined in Eq. 4.6.3.3.2-2.
- Constant depth girders (i.e., girders with parallel flanges – no haunched or tapered girders).
- Superstructure designed to meet the live load deflection limits outlined in Article 2.5.2.6.2.
- Span lengths not exceeding 200 feet.
- No bottom flange lateral bracing.
- Top flange lateral bracing for construction stability (only if needed), limited to near the end of span (see AASHTO LRFD BDS C6.7.4.2.2).
- Non-hybrid girders (i.e., girder flanges and webs in all spans shall be fabricated using steel of the same grade/yield strength).
- Steel grades/yield strengths of 36 ksi or 50 ksi.
- Structural steel using any of the following corrosion protection systems: painted steel, galvanized steel, metallized steel, or uncoated weathering steel.
- Stringer-type cross-section (no girder-substringer systems), with four or more girders in the cross-section.
- No longitudinal web stiffeners.
- Typical round, headed, stud-type shear connectors.
- Cast-in-place concrete composite decks, formed using one of the following methods:

- Conventional (removable) wood or steel forms.
- Stay-in-place corrugated metal forms.
- Partial-depth precast concrete deck panels.

- Routine barrier rail heights (up to 3'-6" tall.)
- No sound walls, noise walls, or other solid barriers atop or adjacent to the barrier rails.
- Bolted field splices.
- Welded steel bearing stiffeners, intermediate stiffeners, and cross-frame/diaphragm connection plates.
- Solid web steel diaphragms (e.g., bent plate, rolled channel shape, rolled I-section, or built-up plate girder diaphragms) bolted to connection plates, or truss-type steel cross-frames bolted or welded to gusset plates or connection plates.
- Routine bearings, i.e., those types of bearings which do not provide restraint of girder major axis bending behavior (bearings which do not restrain girder major axis bending end rotations, do not affect flange stresses, etc.) and thus do not affect the behavior of the superstructure in a manner which would invalidate the results of a typical line girder analysis. Examples of routine bearings include steel-reinforced elastomeric bearing pads, high-load multi-rotational bearings such as disc or pot bearings, or roller and rocker bearings.
- No cover plates.
- Generally Seismic Zone 1 only. The guidance provided in this Design Guide may be useful in the design of bridges in higher seismic zones, with input from a senior bridge engineer with extensive experience in the design of steel girder bridges for high-seismic zones, particularly when it is appropriate to use a spline model or other simplified analysis method for the seismic analysis, such that the need for a seismic analysis model does not imply the need for a refined analysis model of the steel superstructure.
- Highway bridges only. Railroad and transit bridges, and bridge intended for use solely by pedestrians, are excluded from the definition of a "routine steel I-girder bridge" for the purposes of this Guide. However, highway bridges with sidewalk loading are included.
- Bridges for which the superstructure is not subject to stream flow loading, ice loading, or vessel collision loading. This implies, at a minimum, that there is sufficient freeboard between the low chord of the superstructure and the high-water elevation during design flood events.
- No redistribution of negative moments in continuous beam or girder bridges.
- Only single-phase construction or simple multi-phase construction. The definition of "simple multi-phase construction" is as follows: Each phase of construction must meet the

various definitions of a “routine steel I-girder bridge,” including the requirements for overall deck and framing plan geometry and minimum number of girders in the cross-sections of each phase of construction. In addition, a closure bay and closure pour must be provided, whereby erection and deck placement of each independent phase of superstructure construction is completed prior to installation of cross-frames or diaphragms in the closure bay between the adjacent phases of construction, and installation of the closure bay cross-frames or diaphragms is completed prior to placement of the deck closure pour between the adjacent phases of construction. Note that in addition to evaluating each phase of construction, the fully-completed bridge must also be evaluated. For further guidance, see Section 6.3.2.5.4 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For cases which differ from these conditions, consultation with a senior bridge engineer with extensive experience in steel girder bridge design can potentially determine ways to apply some or all of the guidance in this Guide.

- Only full bridge replacement, new bridge construction, or simple bridge widening construction. The definition of “simple bridge widening” is as follows: Both the existing structure (including consideration of any partial demolition or partial removal of existing superstructure) and the widened portion of the superstructure must meet all other definitions of a “routine steel I-girder bridge,” including the requirements for overall deck and framing plan geometry and minimum number of girders in the cross-sections of both the existing superstructure and the widened portion of the superstructure. In addition, a closure bay and closure pour must be provided, whereby erection and deck placement of each independent portion of superstructure construction is completed prior to installation of cross-frames or diaphragms in the closure bay between the adjacent portion of construction, and installation of the closure bay cross-frames or diaphragms is completed prior to placement of the deck closure pour between the adjacent portions of construction. Note that in addition to evaluating each stage of construction, the fully-completed bridge must also be evaluated. For further guidance, see Section 6.3.2.5.4 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For cases which differ from these conditions, consultation with a senior bridge engineer with extensive experience in steel girder bridge design can potentially determine ways to apply some or all of the guidance in this Guide.

These assumptions are intended to limit the scope of the guidance presented in this Guide. Also, by inference many of these assumptions allow for the use of line girder methods of analysis. Refined methods of analysis (such as 2D grid analysis, 2D plate-and-eccentric beam analysis, 3D finite element analysis, etc.) are not required, nor recommended, for the design of routine steel I-girder bridges as defined herein. The use of the empirical live load distribution factors associated with line girder analysis methods usually result in a slightly, but not excessively, more conservative live load distribution than would be determined using a refined method of analysis; nevertheless, refined analysis methods are not required, nor recommended, for the design of routine steel I-girder bridges as defined herein.

Using refined analysis methods for the design of a routine steel I-girder bridge requires additional effort that would be better spent on other tasks such as exploring framing plan and girder design refinement options, improving plan clarity, or implementing more robust checking and QC procedures. Refined analysis methods are also inherently more complex than line girder analysis methods, introducing more opportunities for errors. Finally, and importantly, using a refined method of analysis to decrease conservatism in live load distribution could lead a designer to reduce girder flange or web sizes during the initial design of a bridge. This could result in difficulties later when the Owner-agency performs periodic routine load rating analyses of the bridge. For the sake of practicality, most Owner-agencies default to using line girder analysis methods for these load rating analyses – they have hundreds or thousands of bridges to load rate each year and cannot afford to perform labor-intensive refined analyses when line girder analysis methods would suffice. It is problematic when a bridge exhibits an insufficient load rating due solely to the minor conservatism of line girder analysis methods, forcing the Owner-agency to invest limited resources in performing a refined analysis to demonstrate that a bridge has sufficient load-carrying capacity.

DEFINITION CHECKLIST FOR A "ROUTINE STEEL I-GIRDER BRIDGE"

Answer all questions with "Yes" or "No". If any questions are answered "No", the bridge does not satisfy the definition of a routine steel I-girder bridge for the purposes of this Guide. For further detail on any of these criteria, please see the preceding section of the Guide: DEFINITION OF A "ROUTINE STEEL I-GIRDER BRIDGE"

If a given bridge somehow falls partially outside the limits of the definition of a "routine steel I-girder bridge", or outside the exclusions of this scope, this Guide may still provide value to designers; in such cases, senior bridge engineers with extensive experience in steel bridge design should be consulted when determining if and how to apply any of the recommendations provided herein.

- ☐ Are the girders straight (non-curved) steel I-girders (rolled steel beams or welded plate girders)?
- ☐ Is the deck straight?
- ☐ Is the skew not more than 20 degrees?
- ☐ Are all supports parallel (or within 10 degrees of being parallel)?
- ☐ Are the cross-frames contiguous?
- ☐ Are the girders parallel?
- ☐ Is the deck constant width?
- ☐ Is the Skew Index (Eq. 4.6.3.3.2-2) less than or equal to 0.30?
- ☐ Do the girders have a constant web depth?
- ☐ Is the superstructure designed to meet the live-load deflection limits outlined in Article 2.5.2.6.2?
- ☐ Are all spans less than 200 feet?
- ☐ Design avoids the use of bottom flange lateral bracing?
- ☐ Design uses top flange lateral bracing for construction stability (only if needed), limited to near the end of span (see AASHTO LRFD BDS C6.7.4.2.2)?
- ☐ Are all girder flanges and webs fabricated using steel of the same grade/yield strength?
- ☐ Is the steel grade/yield strength of the girders 36 ksi or 50 ksi?
- ☐ Will one of the following corrosion protection systems – painted steel, galvanized steel, metallized steel, or uncoated weathering steel – be used?
- ☐ Does the bridge feature a stringer-type configuration, with four or more girders in the cross-section?

- ☐ Are the girders to be designed without the use of longitudinal web stiffeners?
- ☐ Are typical round, headed, stud-type shear connectors to be used?
- ☐ Will a cast-in-place composite concrete deck, formed using one of the following methods – conventional (removable) wood or steel forms, stay-in-place corrugated metal forms, partial-depth precast concrete deck panels – be used?
- ☐ Will the bridge be provided with routine barrier rail heights (no more than 3'-6" tall)?
- ☐ Will the bridge be free of superstructure-mounted sound walls, noise walls, or other solid barriers atop or adjacent to the barrier rails?
- ☐ Will any needed field splices be constructed using bolted connections?
- ☐ Will any bearing stiffeners, intermediate stiffeners, and/or cross-frame or diaphragm connection plates be fabricated from welded steel plates or bar stock?
- ☐ Will either solid web steel diaphragms (e.g., bent plate, rolled channel shape, rolled I-section, or built-up plate girder diaphragms) bolted to connection plates, or truss-type steel cross-frames bolted or welded to gusset plates or connection plates, be used to brace the girders?
- ☐ Will only routine bearing types be used?
- ☐ Will the bridge girders be free of cover plates?
- ☐ Is the bridge located in Seismic Zone 1?
- ☐ Is the bridge carrying only highway vehicular traffic?
- ☐ Is the superstructure not subject to stream flow loading, ice loading, and vessel collision loading?
- ☐ Will the use of design methods involving the redistribution of negative moments in continuous beam or girder bridges be avoided?
- ☐ Will the bridge be constructed using only single-phase construction or simple multi-phase construction?
- ☐ Will the construction of the bridge fall into one of these three categories – full bridge replacement, new bridge construction, or simple bridge widening construction?

USEFUL REFERENCES

A list of useful references is provided below. These references are frequently cited in the Guide. Some of the references may be slightly dated but represent the best guidance available at the time of writing of this Guide. In general, the reader can typically compensate for the dated nature of any particular reference by recognizing where references to specific provisions of the AASHTO LRFD BDS may be out of date; in such cases the current provisions of the AAHTO LRFD BDS should be followed and the guidance provided in the reference should be interpreted accordingly.

- AASHTO's AASHTO Guide Specifications for Wind Loads on Bridges During Construction: At the time of the writing of this Guide, the 1st Edition of this guide specification had been published by AASHTO in 2017.
    - https://store.transportation.org/Item/PublicationDetail?ID=3728
- AASHTO's AASHTO LRFD Steel Bridge Fabrication Specifications: At the time of the writing of this Guide, this specification (1st Edition with 2024 Interim Revisions) had been published in 2024.
- AASHTO-NSBA Steel Bridge Collaboration's Guideline G1.4-2006 Guidelines for Design Details: At the time of the writing of this Guide, this guideline had been published by the AASHTO/NSBA Steel Bridge Collaboration in 2006.
    - https://www.aisc.org/globalassets/nsba/aashto-nsba-collab-docs/g-1.4-2006-guidelines-for-design-details.pdf
- AASHTO-NSBA Steel Bridge Collaboration's Guideline G12.1-2020 Guidelines to Design for Constructability and Fabrication: At the time of the writing of this Guide, this guideline had been published by the AASHTO/NSBA Steel Bridge Collaboration in 2020.
    - https://www.aisc.org/globalassets/nsba/aashto-nsba-collab-docs/nsbagdc-3.pdf
- AASHTO-NSBA Steel Bridge Collaboration's Guideline G13.1-2019 Guidelines for Steel Girder Bridge Analysis: At the time of the writing of this Guide, this guideline had been published by the AASHTO/NSBA Steel Bridge Collaboration in 2019.
    - https://www.aisc.org/globalassets/nsba/aashto-nsba-collab-docs/g-13.1-2019-guidelines-for-steel-girder-bridge-analysis.pdf
- AISC's AISC Design Guide 17 High Strength Bolts - A Primer for Engineers: At the time of the writing of this Guide, the 1st Edition of AISC Design Guide 17 had been published in 2002.
    - https://www.aisc.org/products/publication/design-guides/design-guide-17-high-strength-bolts-a-primer-for-structural-engineers/
- AISC's Database of Rolled Steel Shape Section Properties: At the time of the writing of this Guide, Version 16.0 of this database had been released in August 2023.

- https://www.aisc.org/publications/steel-construction-manual-resources/16th-ed-steel-construction-manual/aisc-shapes-database-v16.0/

- AISC's Specifications for Structural Steel Buildings and Commentary: At the time of the writing of this Guide, the current edition of this specification was dated August 1, 2022.

  - https://www.aisc.org/publications/steel-standards/aisc-360/

- ASCE's ASCE 7-10, Minimum Design Loads for Buildings and Other Structures: At the time of the writing of this Guide, this guideline had been published by the American Society of Civil Engineers in 2010. A later version had been published, but the AASHTO wind load provisions are based on the 2010 version.

  - https://sp360.asce.org/PersonifyEbusiness/Merchandise/Product-Details/productId/232961952

- NSBA's Steel Bridge Design Handbook: At the time of the update of this Guide, the NSBA had published this 19 chapter guide, plus 6 full design examples, all based on the 9th Edition of the AASHTO LRFD BDS; the Handbook (and examples) still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS. An updated version based on the 10th Edition of the AASHTO LRFD BDS is in work.

  - https://www.aisc.org/nsba/design-and-estimation-resources/steel-bridge-design-handbook/

- FHWA's FHWA Bridge Welding Reference Manual: At the time of the writing of this Guide, this manual had been published by the FHWA as Report No. FHWA-HIF-19-088, dated September 2019 (with errata September 2020).

  - https://www.fhwa.dot.gov/bridge/steel/pubs/hif19088.pdf

- NHI and FHWA's Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures: At the time of the writing of this Guide, this Reference Manual had been issued by the FHWA as Report No. FHWA-NHI-15-047, dated July 2015. It was based on the 7th Edition, 2014, of the AASHTO LRFD BDS; this Reference Manual contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

  - https://www.fhwa.dot.gov/bridge/pubs/nhi15047.pdf

- NHI and FHWA's Reference Manual for NHI Course 130102, Engineering for Structural Stability in Bridge Construction: At the time of the writing of this Guide, this Reference Manual had been issued by the FHWA as Report No. FHWA-NHI-15-044, dated April 2015. It was based on the 6th Edition, 2012, of the AASHTO LRFD BDS; this Reference Manual contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

- https://www.fhwa.dot.gov/bridge/pubs/nhi15044.pdf

- NHI and FHWA's Reference Manual for NHI Course 130122, Design and Evaluation of Steel Bridges for Fatigue and Fracture: At the time of the writing of this Guide, this Reference Manual had been issued by the FHWA as Report No. FHWA-NHI-16-016, dated December 2016. It was based on the 7th Edition, 2014, of the AASHTO LRFD BDS, with Interim Revisions through 2015; this Reference Manual contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

  - https://www.fhwa.dot.gov/bridge/steel/pubs/nhi16016.pdf

- NSBA's Bolted Field Splices for Steel Bridge Flexural Members – Overview and Design Examples: At the time of the writing of this Guide, the current version of this document was Version 2.03, dated April 2020.

  - https://www.aisc.org/globalassets/nsba/design-resources/nsba-splice/bolted-field-splices-for-steel-bridge-flexural-members.pdf

- NSBA's NSBA Splice Microsoft Excel-based bolted field splice design spreadsheet: At the time of the writing of this Guide, NSBA's Splice Microsoft Excel-based bolted field splice design program functioned in a manner consistent with the 9th Edition, 2020, of the AASHTO LRFD Bridge Design Specifications (AASHTO LRFD BDS).

  - https://www.aisc.org/nsba/design-resources/nsba-splice/

- NSBA's LRFD Simon line-girder analysis and design program: At the time of the writing of this Guide, NSBA's LRFD Simon program functioned in a manner consistent with the 9th Edition, 2024, of the AASHTO LRFD Bridge Design Specifications (AASHTO LRFD BDS).

  - https://www.aisc.org/nsba/design-resources/simon/

- AISC/NSBA's Standard Plans for Steel Bridges: At the time of the writing of this Guide, AISC and NSBA had published Standard Plans for Steel Bridges , which feature designs for steel plate girder bridges covering one-, two-, three-, and four-span configurations with span lengths ranging from 80 feet to 300 feet, and girder spacings of 8, 10, 12, and 14 feet. The designs are consistent with the 10th Edition, 2024, of the AASHTO LRFD BDS.

  - https://www.aisc.org/nsba/design-and-estimation-resources/standard-bridge-plans/

- NSBA's Span-to-Weight Curves: At the time of the writing of this Guide, NSBA's Span-to-Weight curves were based on over based upon over 800 preliminary designs the NSBA has done through the years up to 2016.

  - https://www.aisc.org/nsba/design-resources/span-to-weight-curves/

- The NSBA brief guide to Skewed and Curved Steel I-Girder Bridge Fit (Executive Summary): At the time of the writing of this Guide, NSBA had published a short executive summary on the top of steel I-girder bridge fit.
    - https://www.aisc.org/globalassets/nsba/technical-documents/skewed-curved-steel-bridges-august-2016-summary-final.pdf
- The NSBA in-depth guide to Skewed and Curved Steel I-Girder Bridge Fit (Full White Paper): At the time of the writing of this Guide, NSBA had published an longer, in-depth white-paper on this topic.
    - https://www.aisc.org/globalassets/nsba/technical-documents/skewed-curved-steel-bridges-august-2016-final.pdf
- Short Span Steel Bridge Alliance's Technical Design Resources for Short Span Steel Bridges: At the time of the writing of this Guide, the Short Span Steel Bridge Alliance was posting several design resources at this web page, including access to their eSPAN140 interactive web-based preliminary design aid.
    - https://www.shortspansteelbridges.org/resources/design/
- Research Council on Structural Connections (RCSC)'s Specification for Structural Joints Using High-Strength Bolts: At the time of the writing of this Guide, the current version of this specification was dated June 11, 2020.
    - https://www.aisc.org/publications/steel-standards/rcsc/

GENERAL FLOW OF DESIGN TASKS

Listed below are the general Design Tasks associated with the typical flow of design of a routine steel I-girder bridge superstructure. The list of Design Tasks is presented in roughly the typical order that they occur in the superstructure design process. However, as noted below, some topics apply to several Design Tasks. And, of course, the process of designing a bridge typically involves some degree of iteration; the initial results of later Design Tasks may suggest that revising part of the design which occurred earlier in the process might be beneficial. When iterating through a design in this manner, the designer is reminded that all steps of the design process should be checked to see if the revision of one part of the design might affect other parts. Each task/topic below is hyperlinked to its associated Design Task Quick Links page.

General Flow of Design Tasks:

1. General Considerations
2. Deck Design
3. Resistance Factors and Load Modifiers
4. Load Combinations and Load Factors
5. Live Load Force Effects - Introduction
6. Live Load Force Effects - Flexure
7. Live Load Force Effects - Shear
8. Other Load Effects and Factors Affecting Load Effect Calculations
9. Girder Flexure Design – General
10. Girder Flexure Design – Constructibility
11. Girder Flexure Design – Service Limit State
12. Girder Flexure Design – Fatigue and Fracture Limit State
13. Girder Flexure Design – Strength Limit State
14. Girder Shear Design
15. Stiffener Design
16. Shear Connector Design
17. Splice Design
18. Cross-Frame/Diaphragm Design

Topics Which May Apply to Several Design Tasks:

- Bolted Connection Design
- Welded Connection Design
- Connection Design – Miscellaneous Checks

GRAPHICAL INDEX OF DESIGN TASKS

The general Design Tasks associated with the typical steps in the design of a routine steel I-girder bridge superstructure are grouped graphically below. Each task/topic is hyperlinked to its associated Design Task Quick Links page.

General Considerations

General considerations prior to beginning detailed superstructure design include understanding the LRFD design philosophy and the concept of limit states design, selecting basic design parameters such as target girder depth and spacing, identifying superstructure materials, and deciding whether or not to make the deck composite with the girders. The following Design Tasks apply – each task is hyperlinked to its associated Design Task Quick Links page.

- General Considerations

Deck Design

The design of decks for routine steel I-girder bridges is beyond the scope of this Guide; see the Owner-agencies design policy manual for standard deck designs or guidance on acceptable deck design methods, or design the deck per the provisions of Chapter 9 of the AASHTO LRFD BDS. The following Design Tasks apply – each task is hyperlinked to its associated Design Task Quick Links page.

- Deck Design

Loads

The identification and calculation of various load effects is typically accomplished early in the design process. It is difficult to design the superstructure of a routine steel I-girder bridge if the applicable loads are not known. The process of determining those loads begins with identification of the applicable limit states, definition of their associated load combinations, and quantification of the various load modifiers and load factors. Resistance factors are typically identified and quantified as well. Then the specific loading effects are calculated. The following Design Tasks apply – each task is hyperlinked to its associated Design Task Quick Links page.

- Resistance Factors and Load Modifiers
- Load Combinations and Load Factors
- Live Load Force Effects - Introduction
- Live Load Force Effects - Flexure
- Live Load Force Effects - Shear
- Other Load Effects and Factors Affecting Load Effect Calculations

Girder Design

Once the loads are defined, girder design follows. Various limit states must be addressed in girder flexure design, and the design must reflect the noncomposite or composite nature of the superstructure at the time each load is applied.

Generally, it is most efficient to perform the flexural design first, and then design for shear afterwards. The following Design Tasks apply to girder flexure design – each task is hyperlinked to its associated Design Task Quick Links page.

- Girder Flexure Design – General
- Girder Flexure Design – Constructibility
- Girder Flexure Design – Service Limit State
- Girder Flexure Design – Fatigue and Fracture Limit State
- Girder Flexure Design – Strength Limit State

Once the initial flexure design is completed, shear design of the web follows. It may be appropriate or necessary to iterate back through more than one cycle of flexure design and shear design. The following Design Task applies to girder shear design – the task is hyperlinked to its associated Design Task Quick Links page.

- Girder Shear Design

Design of Details and Bracing

Once the basic girder design is established, design of details and bracing can begin. The design of several details is associated directly with the girders, including stiffener design, shear connector design, and bolted field splice design. The following Design Tasks apply to the design of girder-related details – each task is hyperlinked to its associated Design Task Quick Links page.

- Stiffener Design
- Shear Connector Design
- Splice Design

Next the bracing members (cross-frames or diaphragms) can be designed. The following Design Task applies to bracing design – the task is hyperlinked to its associated Design Task Quick Links page.

- Cross-Frame/Diaphragm Design

Connection Design Topics

Several design topics related to connection design are applicable to one or more Design Tasks. These topics are grouped here for convenience. The following Design Tasks apply to these connection design topics – each task is hyperlinked to its associated Design Task Quick Links page.

- Bolted Connection Design
- Welded Connection Design
- Connection Design – Miscellaneous Checks

DESIGN TASK QUICK LINKS

The design of a routine steel I-girder bridge can be broken down into several tasks, each one quite manageable. Each of these tasks can be made even easier when the designer has access to three things:

- Quick Links to applicable AASHTO LRFD BDS provisions, with Discussion
    - Clicking on the AASHTO LRFD BDS Article number (in parenthesis) will take the reader directly to the Discussion of that Article
- Quick Links to helpful industry design guidelines, references, and examples
    - Clicking on the hyperlink for the given reference will take the reader directly to a free copy of that reference on the Internet
- Quick Links to useful tools
    - Clicking on the hyperlink to the NSBA Simon program or the NSBA Splice spreadsheet will take the reader directly to the page on the NSBA website where these tools can be downloaded for free.

The Quick Links to applicable AASHTO LRFD BDS provisions include hyperlinks to many, but not all, of the Articles for which there are Discussions in this Guide. When looking for the Discussion of a particular Article not cited in the Quick Links, the reader can use the "bookmarks" in the PDF version of the Guide; each Article's Discussion is bookmarked.

GENERAL CONSIDERATIONS

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

General considerations prior to beginning detailed superstructure design include:

- Understanding the LRFD Design Philosophy (1.3.1)
- Understanding LRFD Limit States (1.3.2.1, 1.3.2.2, 1.3.2.3, 1.3.2.4, 1.3.2.5)
- Selecting Composite (6.10.1.1) or Noncomposite (6.10.1.2) Design– ***Routine steel I-girder bridges are composite***.
- Selecting Hybrid or Nonhybrid (6.10.1.3) Design – ***Routine steel I-girder bridges are nonhybrid***
- Selecting Constant or Variable Web Depth (6.10.1.4) – ***Routine steel I-girder bridges use constant web depth***.
- Optional criteria for span-to-depth ratios (2.5.2.6.3)
- Cross-section Proportion Limits (6.10.2.1.1, 6.10.2.2)

Quick links to helpful industry design guidelines, references, and examples

For further background and explanation of general considerations, see:

- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures
  - Sections 1.2 (LRFD Design Philosophy), 1.3 (Limit States), 6.3.3.2 (Girder Depth), 6.3.4 (I-Girder Design and Sizing), 6.3.4.4.5 (Sizing Flanges for Efficient Fabrication), 6.4.2.3.2 (Sections in Positive Flexure), 6.4.2.3.3 (Sections in Negative Flexure), 6.4.2.4.1 (Steel Girder), 6.4.2.4.2 (Concrete Deck), and pages 6.194 and 6.195 (Effects of Creep and Shrinkage)
- NSBA's Steel Bridge Design Handbook
  - Chapter 10: Limit States
  - Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge
  - Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge
  - Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge
- The AASHTO-NSBA Steel Bridge Collaboration Guideline G12.1-2020 Guidelines to Design for Constructability and Fabrication
  - Including: Section 1.5 and Tables 1.4.1.A, 1.4.2.A, and 1.5.2.A

Quick links to useful tools

NSBA's LRFD Simon line-girder analysis and design software. Simon is available for free download from the NSBA website and is also a valuable tool for the design of routine steel I-girder bridges. It calculates loads and resistances in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. It contains a useful web-depth optimization option that automatically generates a series of trial-design input files from an acceptable starting design input file. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available.

Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

DECK DESIGN

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

Design Conventionally Reinforced Concrete Deck – Deck design is beyond the scope of this guide; see Owner-Agency policy manuals for standard deck design if available, or design the deck per chapter 9 of the *AASHTO LRFD BDS*.

Quick links to helpful industry design guidelines, references, and examples

For further background and explanation of deck design, see:

- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures
    - Sections 7 (Decks and Deck Systems), and specifically Section 7.3 (Concrete Deck Slabs), 7.3.1 (General), 7.3.2 (Traditional Design Method), 7.3.3 (Empirical Design Method), and 7.3.4 (Deck Overhang Design).
- NSBA's Steel Bridge Design Handbook
    - Chapter 17: Bridge Deck Design

Quick links to useful tools

N/A

RESISTANCE FACTORS AND LOAD MODIFIERS

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

Select resistance factors for:

- Strength limit state (6.5.4.2)

Select load modifiers for:

- Ductility (1.3.3)
- Redundancy (1.3.4)
- Operational Importance (1.3.5)

Quick links to helpful industry design guidelines, references, and examples

For further background and explanation of resistance factors and load modifiers, see:

- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures
    - Sections 1.2 (LRFD Design Philosophy) and 1.3 (Limit States)
- NSBA's Steel Bridge Design Handbook
    - Chapter 7: Loads and Load Combinations
    - Chapter 10: Limit States

Quick links to useful tools

NSBA's LRFD Simon line-girder analysis and design software. Simon is available for free download from the NSBA website and is also a valuable tool for the design of routine steel I-girder bridges. It calculates loads and resistances in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available.

Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use, including verifying and/or modifying the resistance factors and load modifiers as appropriate.

LOAD COMBINATIONS AND LOAD FACTORS

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

- Select load combinations and load factors (3.4.1) for the following limit states:
    - Strength limit state (6.5.4.1, 6.10.6.1)
    - Service limit state (6.5.2, 6.10.4.2.1)
    - Fatigue and fracture limit state (6.5.3)
    - Constructibility (3.4.2.1, 3.4.2.2, 6.10.3.1)

Quick links to helpful industry design guidelines, references, and examples

For more explanation and examples of the determination of load combinations and load factors, see:

- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures
    - Section 1.3 (Limit States) and Chapter 3 (Loads and Load Factors)
- NSBA's Steel Bridge Design Handbook
    - Chapter 7: Loads and Load Combinations
    - Chapter 10: Limit States

Quick links to useful tools

NSBA's LRFD Simon line-girder analysis and design software. Simon is available for free download from the NSBA website and is also a valuable tool for the design of routine steel I-girder bridges. It calculates loads and resistances in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available.

Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use, including verifying and/or modifying the load combinations and load factors as appropriate.

LIVE LOAD FORCE EFFECTS - INTRODUCTION

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

Calculate live load force effects, which includes consideration of:

- Live Loads (3.6.1.2.1, 3.6.1.2.2, 3.6.1.2.3, 3.6.1.2.4, 3.6.1.3.1, 3.6.1.3.2, 3.6.1.3.3, 3.6.1.4.1, 3.6.1.4.2, 3.6.1.6)
- Number of Lanes (3.6.1.1.1)
- Multiple Presence (3.6.1.1.2)
- Dynamic Load Allowance (3.6.2)

Quick links to helpful industry design guidelines, references, and examples

For more explanation and examples of the determination of live load force effects, see:

- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures (2015)
  - Sections 3.4 (Live Loads) and 4.4 (Live Load)
- NSBA's Steel Bridge Design Handbook
  - Chapter 7: Loads and Load Combinations
  - Chapter 8: Structural Analysis
  - Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge
  - Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge
  - Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge

Quick links to useful tools

NSBA's LRFD Simon line-girder analysis and design software. Simon is available for free download from the NSBA website and is also a valuable tool for the design of routine steel I-girder bridges. It calculates loads and resistances in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Simon automatically calculates force effects on girders due to standard AASHTO LRFD vehicular live load. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available.

Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

LIVE LOAD FORCE EFFECTS - FLEXURE

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

- Determine distribution factors for moment, considering:
  - Interior beams with concrete decks (4.6.2.2.2b)
  - Exterior beams (4.6.2.2.2d)
  - Skewed bridges (4.6.2.2.2e)

Quick links to helpful industry design guidelines, references, and examples

For more explanation and examples of the determination of live load force effects with regards to flexure, see:

- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures
  - Sections 4.4.1 (General), 4.4.2 (Live Load Distribution Factors), 4.4.3 (Influence Lines and Influence Surfaces)
- NSBA's Steel Bridge Design Handbook
  - Chapter 7: Loads and Load Combinations
  - Chapter 8: Structural Analysis
  - Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge
  - Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge
  - Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge

Quick links to useful tools

NSBA's LRFD Simon line-girder analysis and design software. Simon is available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It can automatically calculate the live load distribution factors necessary for the analysis, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available.

Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

LIVE LOAD FORCE EFFECTS - SHEAR

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

- Determine distribution factors for shear, including consideration of:
  - Interior Beams (4.6.2.2.3a)
  - Exterior Beams (4.6.2.2.3b)
  - Skewed Bridges (4.6.2.2.3c)

Quick links to helpful industry design guidelines, references, and examples

For more explanation and examples of the determination of live load force effects with regards to shear, see:

- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures
  - Sections 4.4.1 (General), 4.4.2 (Live Load Distribution Factors), 4.4.3 (Influence Lines and Influence Surfaces)
- NSBA's Steel Bridge Design Handbook
  - Chapter 7: Loads and Load Combinations
  - Chapter 8: Structural Analysis
  - Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge
  - Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge
  - Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge

Quick links to useful tools

NSBA's LRFD Simon line-girder analysis and design software. Simon is available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It can automatically calculate the live load distribution factors necessary for the analysis, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available.

Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

OTHER LOAD EFFECTS AND FACTORS AFFECTING LOAD EFFECT CALCULATIONS

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

Consider other loads (beyond gravity loads such as dead load and live load) which may affect the design of routine steel I-girder bridges. Also consider the effects of the composite concrete deck on the distribution of moment and shear in multispan continuous bridges.

- Consider the effects of the composite concrete deck on the stiffness of multispan continuous bridges (6.10.1.5)
- Consider the effects of wind loading, including flange lateral bending effects (4.6.2.7.1)
- Consider seismic loads (6.16.1, 6.16.3, 3.10.9.2, 4.7.4.1, 4.7.4.2, 4.7.4.3, 4.7.4.4)
- Calculate force effects from other loads such as construction loads (6.10.3.1)

Quick links to helpful industry design guidelines, references, and examples

For more explanation and examples of the determination of other load effects and factors affecting load effect calculation, see:

- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures
  - Sections 3.3 (Construction Loads), 3.5 (Wind Loads), 3.6 (Seismic Loads), 6.5.3 (LRFD Constructibility Design), 6.5.6.5.1 (Wind Loads on I-Sections)
- NSBA's Steel Bridge Design Handbook
  - Chapter 7: Loads and Load Combinations
  - Chapter 8: Structural Analysis
  - Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge
  - Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge
  - Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge

Quick links to useful tools

NSBA's LRFD Simon line-girder analysis and design software. Simon is available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the section properties for the stiffness analysis in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available.

Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

GIRDER FLEXURE DESIGN – GENERAL

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

Design girders for flexure, considering the following general topics:

- Composite Section Stresses (6.10.1.1.1a, 6.10.1.1.1b, 6.10.1.1.1c, 6.10.1.1.1d, 6.10.1.1.1e)
- Flange Stresses and Member Bending Moments (6.10.1.6)
- Fundamental Section Properties (D6.1, D6.2.1, D6.2.2, D6.2.3, D6.3.1, D6.3.2)
- Materials (6.4)
- Material Thickness (6.7.3)

Quick links to helpful industry design guidelines, references, and examples

For more explanation and examples of flexure design, see:

- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures
  - Sections 6.4.5.2 (Plastic Moment), 6.4.5.3 (Yield Moment), 6.4.5.4.1 (Depth of Web in Compression in the Elastic Range), 6.4.5.4.2 (Depth of Web in Compression at the Plastic Moment), and 6.5.2 (LRFD Flexural Design Resistance Equations)
- NSBA's Steel Bridge Design Handbook
  - Chapter 1: Bridge Steels and Their Mechanical Properties
  - Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge
  - Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge
  - Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge
- The Reference Manual for NHI Course 130102, Engineering for Structural Stability in Bridge Construction

In addition, sanity check initial design results by comparing them to NSBA's Span-to-Weight Curves

Quick links to useful tools

NSBA's LRFD Simon line-girder analysis and design software. Simon is available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the stresses in the section in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. NOTE that the Simone software currently does not include the capability to design the girders using the provisions of Appendix A6 to account for the ability of certain compact and noncompact web I-sections to develop flexural resistances significantly greater than the yield moment, M_y. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available.

Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

GIRDER FLEXURE DESIGN – CONSTRUCTIBILITY

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

Design girders for flexure with regards to constructibility, considering the following:

- Constructibility (6.10.3.1, 6.5.4.1), Flowchart (C6.4.1)
- Flexure (6.10.3.2, 6.10.1.8, 6.10.1.9, 6.10.1.10.1, 6.10.8.2, A6.3.3—optional)
- Shear (6.10.3.3)
- Deck placement (6.10.3.4)
- Dead load deflections (6.10.3.5)
- Tension flanges with holes (6.10.1.8)

Quick links to helpful industry design guidelines, references, and examples

For more explanation and examples of flexure design with regards to constructibility, see:

- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures
  - Sections 1.3 (Limit States), 6.4.5.5 (Web Bend Buckling Resistance), 6.5.3 (LRFD Constructibility Design), and 6.5.6 (LRFD Strength Limit State for Flexure)
- NSBA's Steel Bridge Design Handbook
  - Chapter 10: Limit States
  - Chapter 11: Design for Constructability
  - Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge
  - Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge
  - Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge
- The AASHTO-NSBA Steel Bridge Collaboration Guidelines
  - G12.1-2020 Guidelines to Design for Constructability and Fabrication

In addition, sanity check initial design results by comparing them to NSBA's Span-to-Weight Curves

Quick links to useful tools

NSBA's LRFD Simon line-girder analysis and design software. Simon is available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances in accordance with the provisions of the AASHTO LRFD BDS, including the constructibility checks of Article 6.10.3, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available.

Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

GIRDER FLEXURE DESIGN – SERVICE LIMIT STATE

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

Design girders to meet the requirements of the service limit state for flexure, considering the following:

- Service limit state (6.5.2, 6.10.4), Flowchart (C6.4.2)
- Optional live-load deflection limits (2.5.2.6.2) – ***coordinate with Owner-agency policy***
- Elastic deformations (6.10.4.1)
- Permanent deformations (6.10.4.2, 6.10.4.2.1, 6.10.4.2.2)
- Flexure (6.10.1.9.1, 6.10.1.10.1, 6.10.1.10.2)

Quick links to helpful industry design guidelines, references, and examples

For more explanation and examples of flexure design with regards to service limit state, see:

- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures
  - Sections 1.3 (Limit States), 6.4.5.5 (Web Bend Buckling Resistance), and 6.5.4 (LRFD Service Limit State Design)
- NSBA's Steel Bridge Design Handbook
  - Chapter 10: Limit States
  - Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge
  - Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge
  - Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge

In addition, sanity check initial design results by comparing them to NSBA's Span-to-Weight Curves

Quick links to useful tools

NSBA's LRFD Simon line-girder analysis and design software. Simon is available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, including the service limit state checks of Article 6.10.4, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available.

Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

GIRDER FLEXURE DESIGN – FATIGUE AND FRACTURE LIMIT STATE

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

Design girders to meet the requirements of the fatigue and fracture limit state for flexure, considering the following:

- Fatigue and Fracture Limit State (6.5.3, 6.10.5.1, 6.10.5.2), Flowchart (C6.4.3)
- Fatigue (6.6.1.1, 6.6.1.2, 6.6.1.3, 6.10.5.1)
- Fracture (6.6.2.1, 6.10.5.2)
- Special Fatigue Requirement for Webs (6.10.5.3)

Quick links to helpful industry design guidelines, references, and examples

For more explanation and examples of flexure design with regards to fatigue and fracture limit state, see:

- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures
  - Sections 1.3 (Limit States), 6.5.5 (LRFD Fatigue and Fracture Limit State Design), and 6.6.2 (Shear Connectors)
- NSBA's Steel Bridge Design Handbook
  - Chapter 10: Limit States
  - Chapter 12: Design for Fatigue
  - Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge
  - Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge
  - Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge
- The AASHTO-NSBA Steel Bridge Collaboration Guidelines
  - G1.4-2006 Guidelines for Design Details
  - G12.1-2020 Guidelines for Design for Constructabilty

In addition, sanity check initial design results by comparing them to NSBA's Span-to-Weight Curves

Quick links to useful tools

NSBA's LRFD Simon line-girder analysis and design software. Simon is available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. It will calculate the fatigue stress range for either the Fatigue I or Fatigue II limit-state load combination at multiple points along the length of the girder and compare it to the nominal fatigue resistance. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available.

Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

GIRDER FLEXURE DESIGN – STRENGTH LIMIT STATE

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

Design girders for flexure to meet the requirements of the strength limit state, considering the following:

- Strength limit state (6.5.4.1, 6.5.4.2, 6.10.6.1, 6.10.6.2.1), Flowchart (C6.4.4)
- Composite sections in positive flexure (6.10.6.2.2, 6.10.7.1.1, 6.10.7.1.2, 6.10.7.2.2, 6.10.7.3), Flowchart (C6.4.5)
- Composite sections in negative flexure and noncomposite sections (6.10.6.2.3, 6.10.8.1.1, 6.10.8.1.2, 6.10.8.1.3, 6.10.8.2.1, 6.10.8.2.2, 6.10.8.2.3, 0, D6.6), Flowchart (C6.4.6) (APPENDIX A6—optional), Flowchart (C6.4.7−optional) (D6.4—optional)
- Tension flanges with holes (6.10.1.8)
- Flange-strength Reduction Factors (6.10.1.10.1, 6.10.1.10.2)

Quick links to helpful industry design guidelines, references, and examples

For more explanation and examples of flexure design with regards to strength limit state, see:

- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures
    - Sections 1.3 (Limit States) and 6.5.6 (LRFD Strength Limit State Design for Flexure)
- NSBA’s Steel Bridge Design Handbook
    - Chapter 10: Limit States
    - Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge
    - Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge
    - Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge

In addition, sanity check initial design results by comparing them to NSBA’s Span-to-Weight Curves

Quick links to useful tools

NSBA's LRFD Simon line-girder analysis and design software. Simon is available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, including the strength limit state checks of Article 6.10.6, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available.

Users should verify the capabilities, assumptions, and general correctness of any program’s calculations prior to initial use.

GIRDER SHEAR DESIGN

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

Design girders for shear to meet the requirements of the strength limit state, considering the following:

- General provisions (6.10.9.1), Flowchart (Figure C6.10.9.1-1)
- Nominal resistance of unstiffened webs (6.10.9.2)
- Nominal resistance of stiffened webs
    - General provisions (6.10.9.3.1)
    - Nominal resistance of interior panels (6.10.9.3.2)
    - Nominal resistance of end panels (6.10.9.3.3)

Quick links to helpful industry design guidelines, references, and examples

For more explanation and examples of girder shear design, see:

- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures
    - Section 6.5.7 (LRFD Strength Limit State Design for Shear)
- NSBA's Steel Bridge Design Handbook
    - Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge
    - Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge
    - Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge
- The AASHTO-NSBA Steel Bridge Collaboration Guidelines
    - G12.1-2020 Guidelines for Design for Constructabilty

Quick links to useful tools

NSBA's LRFD Simon line-girder analysis and design software. Simon is available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design shear loads, and the corresponding shear resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available.

Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

STIFFENER DESIGN

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

Design stiffeners, considering the following:

- Web transverse stiffener design provisions (6.10.11.1.1, 6.10.11.1.2, 6.10.11.1.3)
- Bearing stiffener design provisions (6.10.11.2.1, 6.10.11.2.2, 6.10.11.2.3, 6.10.11.2.4a, 6.10.11.2.4b)
- Provisions for concentrated loads applied to webs without bearing stiffeners (D6.5.1, D6.5.2, D6.5.3)

Quick links to helpful industry design guidelines, references, and examples

For more explanation and examples of stiffener design, see:

- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures
    - Sections 6.6.6.2 (Transverse Web Stiffeners), 6.6.6.3 (Bearing Stiffeners)
- NSBA's Steel Bridge Design Handbook
    - Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge
    - Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge
- The AASHTO-NSBA Steel Bridge Collaboration Guidelines
    - G1.4-2006 Guidelines for Design Details
    - G12.1-2020 Guidelines to Design for Constructability and Fabricationhttps://www.aisc.org/globalassets/nsba/aashto-nsba-collab-docs/nsbagdc-4.pdf
- The FHWA Bridge Welding Reference Manual

Quick links to useful tools

Various aspects of stiffener design are sometimes addressed in commercial line-girder analysis and design software, but more often the calculations are performed by hand or are automated by designers in spreadsheets. However, even if the detailed design of the stiffeners is performed by hand or spreadsheet, certain design variables such as shear demand and resistance values or bearing reactions are still obtained from the line-girder analysis and design software. NSBA's LRFD Simon line-girder analysis and design software is available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates bearing reactions and shear demand and the corresponding resistances in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available.

Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

SHEAR CONNECTOR DESIGN

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

Design shear connectors for the fatigue and strength limit states, considering the following:

- General provisions (6.10.10.1, 6.10.10.1.1, 6.10.10.1.2, 6.10.10.1.3, 6.10.10.1.4)
- Fatigue resistance (6.10.10.2)
- Special requirements for points of permanent load contraflexure (6.10.10.3)
- Strength limit state (6.10.10.4.1, 6.10.10.4.2, 6.10.10.4.3)

Quick links to helpful industry design guidelines, references, and examples

For more explanation and examples of the determination of the design of shear connectors at the fatigue and strength limit states, see:

- The Reference Manual for NHI Course 130122, Design and Evaluation of Steel Bridges for Fatigue and Fracture
  - Section 6.3.6.3 (Shear Studs)
- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures
  - Section 6.6.2 (Shear Connectors)
- NSBA's Steel Bridge Design Handbook
  - Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge
  - Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge
  - Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge

Quick links to useful tools

NSBA's LRFD Simon line-girder analysis and design software. Simon is available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It performs design calculations addressing the demand on, and resistance of, shear connectors at the fatigue and strength limit states in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available.

Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

SPLICE DESIGN

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

Design field splices (if present), considering the following:

- Bolted field splices of flexural members
  - General considerations (6.13.6.1.3a)
  - Flange splices (6.13.6.1.3b)
  - Web splices (6.13.6.1.3c)
- Welded splices (6.13.6.2)
- Minimum thickness requirements (6.7.3)

Determine flange sizes and locations of welded shop splices, considering the following:

- Welded splices (6.13.6.2)
- Minimum thickness requirements (6.7.3)

Quick links to helpful industry design guidelines, references, and examples

For more explanation and examples of field splice design, see:

- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures
  - Sections 6.6.5 (Splices), especially 6.6.5.2 (Flexural Members) (NOTE: The explanations in these references are written in the context of the bolted field splice provisions prior to publication of the 8th Edition of the AASHTO LRFD BDS and are thus out of date).
- The AASHTO-NSBA Steel Bridge Collaboration Guidelines G12.1-2020 Guidelines to Design for Constructability and Fabrication
  - Section 1.5.3 (Flange Plate Width) and Table 1.5.2.A, Section 2.2.1 (Field Connections)
- NSBA's Bolted Field Splices for Steel Bridge Flexural Members – Overview and Design Examples

Quick links to useful tools

The NSBA Splice Microsoft Excel-based bolted field splice design spreadsheet is available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It performs the design of a bolted field splice for a steel I-girder in accordance with the provisions of Article 6.13.6.1.3, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to design bolted field splices are also available.

Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

CROSS-FRAME/DIAPHRAGM DESIGN

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

Design cross-frames or diaphragms, considering the following:

- General considerations (6.7.4.1, 6.7.4.2), minimum thicknesses (6.7.3)
- Design for tension (6.8.1, 6.8.2.1, 6.8.2.2, 6.8.3, 6.8.4)
- Design for compression (6.9.1, 6.9.2.1, 6.9.2.2.1, 6.9.3, 6.9.4.1.1, 6.9.4.1.2, 6.9.4.1.3, 6.9.4.2.1, 6.9.4.2.2a, 6.9.4.2.2b, 6.9.4.4)
- Design considerations for miscellaneous flexural members (6.12.1.1, 6.12.1.2.1, 6.12.1.2.2, 6.12.1.2.3a, 6.12.2.1, 6.12.2.2.4a, 6.12.2.2.4b, **Error! Reference source not found.**, 6.12.2.2.4d, 6.12.2.2.4e, 6.12.2.2.5)

Quick links to helpful industry design guidelines, references, and examples

For more explanation and examples of the determination of the design of shear connectors at the fatigue and strength limit states, see:

- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures
    - Section 6.6.3 (Bracing Member Design)
- NSBA's Steel Bridge Design Handbook
    - Chapter 13: Bracing System Design
    - Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge
- The AASHTO-NSBA Steel Bridge Collaboration Guidelines
    - G1.4-2006 Guidelines for Design Details
    - G12.1-2020 Guidelines to Design for Constructability and Fabrication
- AISC's Specifications for Structural Steel Buildings and Commentary
    - Article D3, Table D3.1, and the Commentary for Article D3
- AISC's Database of Rolled Steel Shape Section Properties

Quick links to useful tools

The calculations associated with cross-frame and diaphragm design for routine steel I-girder bridges are typically performed by hand or in spreadsheets. Some commercial bridge design software packages offer some capabilities associated with cross-frame or diaphragm design, but those capabilities are typically limited to refined analysis software packages, not line girder analysis and design packages.

Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

BOLTED CONNECTION DESIGN

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

Design bolted connections for cross-frames or diaphragms and for bolted field splices, considering the following (also see the Quick Links for field splice design):

- General provisions (6.13.1, 6.13.2.1)
- Bolt, nut, washer, and bolt hole provisions (6.13.2.3.1, 6.13.2.3.2, 6.13.2.4.1a, 6.13.2.4.1b, 6.13.2.4.1c, 6.13.2.4.1d, 6.13.2.4.2, 6.13.2.5)
- Bolt spacing, edge and end distances (6.13.2.6.1, 6.13.2.6.2, 6.13.2.6.3, 6.13.2.6.4, 6.13.2.6.5, 6.13.2.6.6)
- Net area (6.8.3)
- Factored resistance of bolted connections (6.13.2.2)
- Slip critical bolt resistance (6.13.2.1.1) (6.13.2.8)
- Bearing connections (6.13.2.1.2), bolt shear resistance (6.13.2.7), bearing resistance at bolt holes (6.13.2.9)
- Bolt tensile resistance (6.13.2.10.1, 6.13.2.10.2, 6.13.2.10.3, 6.13.2.10.4, 6.13.2.11)

Quick links to helpful industry design guidelines, references, and examples

For more explanation and examples of bolted connection design, see:

- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures
    - Sections 6.6.3.2 (Stability Bracing Requirements), 6.6.3.3.2.3 (Net Area), 6.6.4.2 (Bolted Connections) and associated subsections
- The AISC Design Guide 17 High Strength Bolts - A Primer for Engineers
- The RCSC Specifications for Structural Joints Using High-Strength Bolts
- The AASHTO-NSBA Steel Bridge Collaboration Guideline G12.1-2020 Guidelines to Design for Constructability and Fabrication
- The AASHTO LRFD Steel Bridge Fabrication Specifications

Quick links to useful tools

The calculations associated with most bolted connection designs (including cross-frame bolted connections and general bolted connections) are typically performed by hand or in spreadsheets.

See also the Design Task Quick Links for Field Splice Design for reference to the NSBA Splice spreadsheet.

WELDED CONNECTION DESIGN

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

Design welded connections considering the following:

- General provisions (6.13.3.16.13.3)
- Factored resistance (6.13.3.2.1)
- Complete joint penetration welded connections (6.13.3.2.2a, 6.13.3.2.2b)
- Partial penetration groove-welded connections (6.13.3.2.3a, 6.13.3.2.3b)
- Fillet-welded connections (6.13.3.2.4, 6.13.3.4, 6.13.3.5, 6.13.3.6, 6.13.3.7)
- Effective area (6.13.3.3)

Quick links to helpful industry design guidelines, references, and examples

For more explanation and examples of weld connection design, see:

- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures
  - Sections 6.6.4.3 (Welded Connections)
- The AASHTO-NSBA Steel Bridge Collaboration Guidelines
  - G1.4-2006 Guidelines for Design Details
  - G12.1-2020 Guidelines to Design for Constructability and Fabrication
- The FHWA Bridge Welding Reference Manual

Quick links to useful tools

The calculations associated with welded connection design are typically performed by hand or in spreadsheets.

CONNECTION DESIGN – MISCELLANEOUS CHECKS

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

Evaluate all connections (including elements in bolted and welded connections such as splice plates, gusset plates, brackets, etc.) for the following considerations:

- Block shear rupture resistance (6.13.4)
- Connection elements - tension (6.13.5.2)
- Connection elements - shear (6.13.5.3)

Quick links to helpful industry design guidelines, references, and examples

For more explanation and examples of miscellaneous checks of connection design, see:

- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures
    - Sections 6.6.3.3.2.5 (Block Shear Rupture Resistance), 6.6.4.2.5.6.1 (Tensile Resistance of a Connected Element), and 6.6.4.2.5.6.2 (Shear Resistance of a Connected Element)
- NSBA's Bolted Field Splices for Steel Bridge Flexural Members – Overview and Design Examples

Quick links to useful tools

The calculations associated with miscellaneous connection design are typically performed by hand or in spreadsheets.

See also the Design Task Quick Links for Field Splice Design for reference to the NSBA Splice spreadsheet.

SECTION 1: INTRODUCTION

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1.1 SCOPE OF THE SPECIFICATIONS

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article discusses the overall scope of the AASHTO LRFD BDS, and from that perspective is fairly self-explanatory. Key guidance includes the statement, "These Specifications are not intended to supplant proper training or the exercise of judgment by the Designer, and state only the minimum requirements necessary to provide for public safety." In a similar vein, this "Guide for streamlined design of routine steel I-girder bridges" is also only another tool. Ultimately, the design of any bridge should be performed, or at least directly supervised, by engineers who are experienced and qualified to do the work. No amount of specifications, commentary, guidelines, or pre-packaged design software can take the place of proper training, experience, and oversight.

The Article continues by discussing basic concepts upon which the AASHTO LRFD BDS are based, and providing references to associated specifications which cover topics not directly addressed in the AASHTO LRFD BDS.

The Commentary for this Article provides important definitions of the terms "notional," "shall," "should," and "may," which have specific connotations throughout the specifications.

1.2 DEFINITIONS

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The terms listed are either directly applicable to, or help define provisions which are not applicable to, the design of routine steel I-girder bridges.

1.3 DESIGN PHILOSOPHY

1.3.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The design philosophy of the AASHTO LRFD BDS encompasses the design of routine steel I-girder bridges, among other structures.

1.3.2 Limit States

1.3.2.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The basic load and resistance factor design (LRFD) equation and the associated equations in this Article limiting the combined values of the load modifiers apply to all bridges, including the routine steel I-girder bridges covered by this Guide. The basic LRFD equation is intended to check that the force effects caused by factored loads do not exceed the factored resistance of the component under consideration. The specified load and resistance factors are statistically based to provide a targeted level of reliability, or probability of exceedance of a given limit state, over the 75-year design life of the bridge. A resistance factor of 1.0 is typically applied to the nominal resistance at all non-strength limit states unless otherwise specified.

The load modifier in the basic equation is a factor that is used in a subjective fashion to account for the ductility, redundancy, and operational classification of the bridge (see the Discussion of Articles 1.3.3, 1.3.4, and 1.3.5 in this Guide). For a routine steel I-girder bridge, the load modifiers for ductility and redundancy are only applicable at the strength limit state; their value is taken as 1.0 at other limit states. Also, Eq. 1.3.2.1-3 is only applicable for the calculation of the load modifier when dead- and live-load force effects are of opposite sign and the minimum load factor specified in Table 3.4.1-2 is applied to the dead-load force effects (e.g., when investigating for uplift at a support or when designing bolted field splices located near points of permanent load contraflexure); otherwise, Eq. 1.3.2.1-2 is to be used.

A limit state is defined as a condition beyond which the bridge or component ceases to satisfy the provisions for which it was designed. The various limit states defined in the specifications, which are considered of equal importance, were established to allow categorization of the evaluation of different combinations of loads (including both unfactored and factored loads) and corresponding resistance values representing different aspects of structural performance. In general, structures are required to provide different levels of performance for routine, frequent loading conditions versus infrequent or extreme loading conditions.

For further background and explanation of the LRFD design philosophy and the basic LRFD design equation, consult Section 1.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For further background and explanation of limit states, consult the NSBA's Steel Bridge Design Handbook – Chapter 10: Limit States, and Section 1.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

1.3.2.2 Service Limit State

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

All steel girder bridges, including the routine steel I-girder bridges covered by this Guide, are subject to limits on stresses and deformations under service limit state loading conditions, which generally refers to normal operational use of the bridge under service conditions. The load factors for the service limit state are generally (but not always) set at 1.0.

Steel bridges are not subject to specific limits on crack widths, which in the context of this Article refer to the width of cracks in concrete structures, not fatigue cracks in steel structures.

For further background and explanation of the service limit state, consult the NSBA's Steel Bridge Design Handbook – Chapter 10: Limit States, and Section 1.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

1.3.2.3 Fatigue and Fracture Limit State

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

Fatigue is defined in Section 6 of the AASHTO LRFD BDS as, "The initiation and/or propagation of cracks due to a repeated variation of normal stress with a tensile component." Fracture is the partial or total severing of an element under the action of force, particularly a tensile force. Fatigue and fracture are phenomena which can easily occur in an improperly designed or detailed steel structure. The fatigue and fracture limit state is directly applicable to the design of all steel bridges, including the routine steel I-girder bridges covered by this Guide.

The discussion of cracks in the commentary refers to fatigue cracks in steel structures, not tension or shear cracks in concrete structures.

For further background and explanation of the fatigue and fracture limit state, consult the NSBA's Steel Bridge Design Handbook – Chapter 10: Limit States, FHWA's Reference Manual for NHI Course 130122, Design and Evaluation of Steel Bridges for Fatigue and Fracture, and Section 1.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

1.3.2.4 Strength Limit State

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The strength limit state addresses the investigation of stability and/or yielding of structural elements. The strength limit state is applicable to the design of all structures, including the routine steel I-girder bridges covered by this Guide.

For further background and explanation of the strength limit state, consult the NSBA's Steel Bridge Design Handbook – Chapter 10: Limit States, and Section 1.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

1.3.2.5 Extreme Event Limit State

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The extreme event limit state applies primarily to substructure design, which is excluded from the scope of this Guide. The rare cases of applicability of the extreme event limit state to the design of steel I-girder bridge superstructures are limited to situations which have been excluded from the definition given herein of a routine steel I-girder bridge. For instance, in bridges subject to higher seismic demands (Seismic Zones 2, 3, or 4 as defined in the AASHTO LRFD BDS), the design of pier and end cross-frames or diaphragms may constitute part of the load path transmitting seismic loads from the superstructure to the substructure under the extreme event limit state for seismic design. However, the definition of a routine steel I-girder bridge for the purposes of this Guide only included bridges in Seismic Zone 1.

1.3.3 Ductility

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The structural members and details used in the design of routine steel I-girder bridge superstructures are specifically designed and configured to inherently exhibit ductile behavior, so the specified value of 1.0 for the load modifier for ductility for conventional designs and details should always be used for the design of routine steel I-girder bridge superstructures.

The use of a load modifier for ductility with a value other than 1.0 may be appropriate for design of other elements in a given bridge, such as the substructure. In such cases, designers are advised to carefully identify and differentiate elements which are subject to the application of such a ductility load modifier in their design.

1.3.4 Redundancy

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The configuration, associated structural member designs, and details of a routine steel I-girder bridge, as defined for the purposes of this Guide, inherently provide multiple, redundant load paths, in both simple span and multiple-span continuous bridges. Therefore, the specified value of 1.0 for the load modifier for redundancy for conventional levels of redundancy should always be used for the design of routine steel I-girder bridge superstructures.

1.3.5 Operational Importance

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The value of the load modifier for operational importance should be chosen based on a careful evaluation of a given bridge in the larger context of the transportation network in which it functions. Generally, this evaluation is performed by the Owner-agency or their designated representative. Alternately, the Owner-agency or their designated representative may provide explicit guidance on how to perform such an evaluation for bridges within their transportation network. In either case, the value of the load modifier for operational importance should be chosen with input from the Owner-agency. In the absence of such input, the load modifier for operational importance should be taken as 1.0.

1.4 REFERENCES

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

Helpful reference material is provided for more in-depth discussion of the various provisions within Section 1 of the AASHTO LRFD BDS.

SECTION 2: GENERAL DESIGN AND LOCATION FEATURES

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2.1 SCOPE .... *not addressed in this Guide*
2.2 DEFINITIONS .... *not addressed in this Guide*
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2.5.2.6 Deformations

2.5.2.6.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

Only the first paragraph of the provision is applicable to the routine steel I-girder bridges covered by this Guide. The remainder of the provisions in this article related to dynamic analysis and to straight skewed and horizontally curved steel girder bridges are not applicable to these bridges.

2.5.2.6.2 Criteria for Deflection

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

These criteria are optional at the direction of the Owner-agency; review Owner-agency guidelines in conjunction with the provisions of this Article. In the absence of Owner-agency guidelines, design routine steel I-girder bridges to meet the applicable live load deflection limits presented in this article for "steel, aluminum, and/or concrete vehicular bridges." Only the provisions directly related to straight steel I-girder bridges apply to the design of the routine steel I-girder bridges covered by this Guide.

Designs in which the live load deflections of the superstructure exceed the limits suggested in this Article may be at risk of experiencing adverse dynamic response (vibrations) under routine service loading. The use of dynamic analysis to investigate and/or substantiate the dynamic performance of a design in which the live load deflections exceed these limits is not recommended for routine steel I-girder bridges.

Furthermore, designs which do not meet these simple live load deflection limits are generally uneconomical, may be difficult to construct, and/or may have difficulty meeting other design criteria.

The special live load specified in Article 3.6.1.3.2 should be used to evaluate live load deflection. The specified load is intended to produce live load deflections similar to those produced by HS20 loading, which was the basic design live load specified in the AASHTO *Standard Specifications for Highway Bridges* (see the Discussion of Article 3.6.1.3.2 in this Guide). The load factor on the special live load is taken equal to 1.0. For the routine steel I-girder bridges covered by this Guide, all design lanes should be loaded and the beams and girders should be assumed to deflect equally; for multi-girder bridges, this is equivalent to saying that the distribution factor for calculating live load deflection should be taken equal to the appropriate multiple presence factor, *m*, given in Table 3.6.1.1.2-1 (see the Discussion of Article 3.6.1.1.2 in this Guide) times the corresponding number of design lanes loaded divided by the number of girders in the cross-section. The live load deflection is typically limited to $L/800$ for bridges carrying vehicular loading only and $L/1000$ for bridges carrying vehicular and pedestrian loading, where L is the span length in feet.

Concrete barriers and sidewalks, and even railings, often contribute to the stiffness of composite superstructures at service load levels. Therefore, this Article permits the entire width of the roadway and the structurally continuous portions of railings, sidewalks and barriers (i.e., continuous cast-in-place barriers) to be included in determining the composite stiffness for deflection calculations. However, including concrete items other than the deck complicates the calculation of the composite stiffness of the superstructure and is virtually never done in the design of routine steel I-girder bridges. Barriers are generally located at the edges of the deck, where they tend to stiffen and draw load to the exterior girders. Thus, any beneficial stiffening of the system tends to be counterbalanced by unequal distribution of the loading among the girders and the associated reduction in computed deflections resulting from consideration of the barriers tends to be negligible.

For further discussion on span-to-depth ratios and live load deflection, consult Sections 6.3.3.1 and 6.5.4.2.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples illustrating span-to-depth ratio and live load deflection calculations, consult NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

2.5.2.6.3 Optional Criteria for Span-to-Depth Ratios

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

These criteria are optional at the direction of the Owner-agency; review Owner-agency guidelines in conjunction with the provisions of this Article. In the absence of Owner-agency guidelines, following the guidance on minimum superstructure depths presented in Table 2.5.2.6.3-1 for steel I-beam bridges is recommended, and typically results in more economical, better-performing designs. Only the provisions directly related to straight steel I-girder bridges apply to routine steel I-girder bridges.

To help control elastic deformations at the service limit state, the optional span-to-depth ratios given for a constant-depth superstructure in Table 2.5.2.6.3-1 should be met for all routine steel I-girder bridges to establish a reasonable minimum web depth for the design in the absence of specific depth restrictions. Shallower girders may be used when clearance limits girder depth if permitted by the Owner-agency. For continuous spans, the suggested limits include a built-in factor of 0.8 to reflect an effective span length based on an approximate distance within the span between points of permanent load contraflexure. Typically, the longest span length is used to establish the limit. For end spans, a depth-to-span ratio of 0.9 of the simple-span ratio might be considered to better account for only one end of the span being restrained by continuity. Although the suggested minimum depths are taken to apply to the overall depth of the steel girder, it is suggested that they be applied to the web depth for simplicity. The greatest depth determined from the applicable equation for each span in a continuous girder should be used. Girder depths at, or most often exceeding, these suggested minimum depths typically provide the most economical girders. In many cases, the optimum web depth (see the Discussion of Article 6.10.2.1.1 in this Guide) will be somewhat greater than the minimum depth based on the traditional span-to-depth ratios. The provisions in this Article regarding the suggested minimum span-to-depth ratios for curved steel girder systems do not apply to the routine steel I-girder bridges covered by this Guide.

As noted in the AASHTO-NSBA Steel Bridge Collaboration Guideline G12.1-2020 Guidelines to Design for Constructability and Fabrication (see commentary section C1.5.3), "Deeper girders are generally more economical, but only up to a point. To assess overall economy, it may be valuable to perform a web depth study where the web depth is incrementally increased, the girder is redesigned (targeting a partially stiffened web design), and the resulting girder weight versus depth is recorded. These data points (girder weight versus web depth) can then be plotted to determine the optimum (minimum girder weight) web depth. Some steel girder design software packages (e.g., NSBA's LRFD Simon line-girder analysis and design program) offer automated web depth study features; otherwise the study can be performed by simply iterating the design with different web depths." Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use. Other industry guideline documents present similar recommendations. See the Discussions of Articles 6.10.4.1 and 6.10.2.1.1 in this Guide for further information on the computation of span-to-depth ratios and on web-depth optimization, respectively, for routine steel I-girder bridges.

For further information on span-to-depth ratios, consult Section 6.3.3.1 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples illustrating span-to-depth ratio calculations, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

SECTION 3: LOADS AND LOAD FACTORS

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3.4 LOAD FACTORS AND COMBINATIONS

3.4.1 Load Factors and Load Combinations

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article defines the factored load combinations necessary for bridge design. The limit-state load combinations and associated load factors are specified in Table 3.4.1-1. A number of limit-state load combinations apply to the design of routine steel I-girder bridge superstructures. Strength I and II apply. Strength III and V also apply, but only to the analysis of interim construction conditions and to cross-frame and substructure design. Service I and II apply. Fatigue I applies when infinite fatigue life design is required per Article 6.6.1.2.3; otherwise, Fatigue II applies for finite fatigue life. See the Discussion of Articles 3.6.1.4.2 and 6.6.1.2.3 in this Guide for more guidance on the Fatigue I and II limit-state load combinations.

Conversely, a number of limit-state load combinations do not apply to, and should not be considered in, the design of the routine steel I-girder bridges covered by this Guide. Strength IV is intended to address structures with a high dead load to live load ratio exceeding 7.0, but the routine steel I-girder bridges covered by this Guide do not exhibit such high dead load to live load ratios. Extreme Event I may apply in areas subject to significant seismic effects (Seismic Zones 2, 3, and 4), but design for those types of conditions is considered beyond the definition given herein of a routine steel I-girder bridge. Extreme Event I may apply in the design of bridges subject to less severe seismic effects (Seismic Zone 1), but only to bearing and substructure design, which are beyond the scope of this Guide. Service III and IV are specifically applicable only to the design of segmental concrete girders and concrete substructures, respectively, and thus do not apply to the design of the routine steel I-girder bridges covered by this Guide.

The Extreme Event II limit state may apply to the design of routine steel I-girder bridge superstructures in a limited way; specifically, it may apply to the design of deck overhangs when the barrier rail on the deck overhang is investigated for vehicular collision load, CT. In many such cases, however, the design of the deck overhang may have already been determined and documented in the Owner-agency's standard deck design details or standard deck design tables, and thus may not require a unique design investigation. Other than this case of limited applicability to deck overhang design, the Extreme Event II limit state generally only applies on a case-by-case basis and only bridge substructure components in situations when they may be subjected to ice loads, blast loads, collision by vessels or vehicles, or check floods.

Only the permanent loads DC and DW and the transient loads IM, LL, WL, and WS, as defined in Article 3.3.2, are applicable to the design of the routine steel I-girder bridges covered by this Guide. The permanent loads CR, DD, EH, EL, ES, EV and PS and the transient loads BL, BR, CE, CT (except as discussed above), CV, EQ, FR, IC, LS, SE, TG, TU, and WA do not affect the structural design of superstructures and should not be considered in the design of routine steel I-girder bridge superstructures.

In Table 3.4.1-1, the load factors for the permanent loads DC and DW in the strength limit state load combinations are not provided with singular numeric values, but instead are designated with a variable, γ_p, to allow the designation of minimum and maximum values. This addresses the need to consider potential variability in these loads, where a minimum value might create a certain critical loading condition, while a maximum value might create a different critical loading condition. The values of γ_p for the permanent loads DC and DW are specified in Table 3.4.1-2; other load factors specified in Table 3.4.1-2 do not affect the design of superstructures. Also, only the load factors for DC specified on the first line of the table are applicable for routine steel I-girder bridge superstructures (i.e., the load factors specified for Strength IV do not apply). The specified minimum load factors for DC and DW in Table 3.4.1-2 are only to be applied when the dead- and live-load force effects are of opposite sign; e.g., when investigating for uplift at end supports or when designing bolted field splices located near points of permanent load contraflexure. The load factors specified in Tables 3.4.1-3 through 3.4.1-5 are not applicable for the design of routine steel I-girder bridge superstructures.

For further discussion on loads and limit-state load combinations, consult Chapter 3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

3.4.2 Load Factors for Construction Loads

3.4.2.1 Evaluation at the Strength Limit State

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article specifies the minimum required load factors for constructibility checks. For routine steel I-girder design, the constructibility checks apply to the timeframe prior to when the concrete deck is cured (i.e., the final structural condition).

The standard of care in many jurisdictions is that the designer need only perform a non-structural review of the conceptual erection sequence for the structural steel framing, primarily to demonstrate that a viable erection scheme exists (i.e., an erection sequence that is feasible given the known site conditions and constraints, specified maintenance-of-traffic sequence and requirements, etc.), including consideration of the location of shoring towers, lifting and holding cranes, etc. Owner-agencies in these jurisdictions expect detailed erection engineering to be performed by the Contractor's engineer, not by the bridge's designer. However, some Owner-agencies do require that the designer perform some level of detailed erection engineering. Review local Owner-agency design policies and construction specifications and the local standard of care to determine the requirements in any given specific jurisdiction. Note that the performance of detailed erection engineering is beyond the scope of this Guide.

Once the structural steel framing system is fully erected, the designer is responsible for checking that the structural steel has sufficient strength and stiffness to resist construction loads. The noncomposite steel superstructure should be evaluated for constructibility using the load factors prescribed in this Article. The constructibility checks typically involve more than just consideration of the weight of the wet concrete deck on the noncomposite girders, which is usually applied sequentially, but also construction equipment and worker loads, wind loads, and deck overhang falsework and formwork loads. The load factors and load combinations used for these constructibility checks are specified in this article and include load combinations for the Strength I and Strength III limit states, as well as a special load combination discussed in the last paragraph of the article, in which a load factor of 1.4 is applied to the DC and construction loads. This special load combination typically controls the constructibility checks, but all of the specified load combinations must be investigated. The specified load factors for Strength III involve design checks for wind loads acting on the fully erected steelwork (see the Discussion of Articles 3.8 and 4.6.2.7.3 in this Guide).

For the design constructibility checks, the positive moment regions of the exterior (fascia) girders in their noncomposite condition are typically evaluated for the combined effects of the self-weight of the girders, the sequential deck-casting sequence, and the deck overhang and wind loads (as appropriate) for a "construction active" case, and self-weight plus wind load for a "construction inactive" case. Consult Owner-agency policy with regard to specific loads and load combinations to be investigated during construction. The constructibility checks generally control the design of the top flange of the girder and the cross-frame or diaphragm spacing in the positive moment region. Most commercial steel bridge design software will perform a sequential deck-casting analysis but may not necessarily evaluate the additional effects of deck overhang and wind loads acting on the exterior girders; therefore, the combined effects may need to be evaluated through separate computations.

See the Discussion of Article 6.10.3 in this Guide for more detailed information on the required design constructibility checks utilizing the load factors specified in this Article. The Reference Manual for NHI Course 130102, Engineering for Structural Stability in Bridge Construction provides further discussion of constructibility checks and the associated loads. The reader is cautioned that the Reference Manual for NHI Course 130102 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS. In addition, the AASHTO *Guide Specifications for Wind Loads on Bridges During Construction*, presents wind load provisions addressing wind loads that may occur during the length of time between erection of girders and placement of the deck, and can be consulted in the absence of Owner-agency guidance.

3.4.2.2 Evaluation of Deflection at the Service Limit State

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article specifies load factors for evaluating deflections during construction. In terms of vertical deflections, this Article does not typically apply to routine steel I-girder bridges unless site constraints or contract requirements limit superstructure deflections under construction loading. In terms of horizontal deflections, several Owner-agencies require evaluation of lateral deflections of the steel framing under wind loads; check local Owner-agency policy to see if this requirement applies. The wind pressure used for checking deflections during construction is usually of lower magnitude than the wind loading applied to the final structure; this reflects the shorter duration of construction vs. the anticipated service life of the bridge and thus the lower probability of an extreme storm event.

These checks are generally performed using fairly simple calculations and are generally performed by hand or programmed into a simple spreadsheet. Typically, the routine steel I-girder bridges covered by this Guide will pass these checks due to their relatively limited span lengths. In some cases, slightly exceeding the lateral deflection limit when checking a routine steel I-girder bridges under wind loading may be a sign that the flanges are too narrow; consider using wider flanges as the initial step to address the situation. In cases where the deflections are more significant, the use of a limited amount of top flange lateral bracing near the ends of the span may be a more effective and practical solution, particularly in longer single span bridges (single span structures with lengths near the upper end of the 200-foot span length limit of the routine steel I-girder bridges considered in this Guide). The introduction of lateral bracing to control horizontal deflections prior to construction of the composite concrete decks adds complexity to the design and construction and is beyond the scope of this Guide.

Section 7.4.4 of the Reference Manual for NHI Course 130102, Engineering for Structural Stability in Bridge Construction provides a good discussion of the calculation of horizontal wind loading on the non-composite structural steel framing. Section 6.5.3.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures provides a good discussion of the evaluation of horizontal wind loading on the non-composite structural steel framing, including a simplified example calculation. NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge also provides a good discussion of the evaluation of horizontal wind loading on the non-composite structural steel framing, including a simplified example calculation. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS. In addition, the AASHTO *Guide Specifications for Wind Loads on Bridges During Construction*, presents wind load provisions addressing wind loads that may occur during the length of time between erection of girders and placement of the deck, and can be consulted in the absence of Owner-agency guidance.

3.4.3 Load Factors for Jacking and Post-Tensioning Forces

3.4.3.1 Jacking Forces

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article specifies the minimum load factors for dead load and live load associated with the design of elements affected by future jacking of the bridge (for example, to replace bearings) while the bridge remains in service. This Article is only applicable when the Owner-agency requires that the bridge be designed to accommodate future jacking. In these situations, the designer first determines appropriate locations for the placement of the jacks, and then must design or check the structural elements that will be affected by the loads applied by the jacks when lifting the bridge. Depending on the method and location of jacking, the girders and/or cross-frames may be affected, as well as the substructures. Design details to address jacking loads, including items such as jacking diaphragms, jacking stiffeners on the girder or diaphragms at supports, etc., may need to be added.

3.4.3.2 Force for Post-Tensioning Anchorage Zones

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies the load factor for post-tensioning anchor zones. The routine steel I-girder bridges covered by this Guide do not contain post-tensioning; therefore, this Article is not applicable.

3.4.4 Load Factors for Orthotropic Decks

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies additional fatigue load factors for the design orthotropic decks. The routine steel I-girder bridges covered by this Guide do not use orthotropic decks; therefore, this Article is not applicable.

3.4.5 Load Factors for Cross-Frames and Diaphragms at the Fatigue Limit State

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article presents a factor that is to be multiplied by the Fatigue I and II live load factors, γ_{LL}, when evaluating load-induced fatigue in cross-frames and diaphragms. This provision, based on comprehensive research discussed in the AASHTO LRFD BDS Commentary (C3.4.5), is extremely useful for determining realistic fatigue stress ranges in cross-frames and diaphragms of steel girder bridges, but only applies to bridges with significant support skew and/or horizontal curvature; those types of bridges should be analyzed using a refined analysis method as discussed in Article 4.6.3.3. The routine steel I-girder bridges that are the subject of this Guide feature straight girders with little or no support skew and are (and should be) designed using line girder analysis methods which do not evaluate cross-frame or diaphragm forces or fatigue stress ranges. Cross-frames and diaphragms in the routine steel I-girder bridges addressed in this Guide do not need to be checked for fatigue; the research discussed in AASHTO LRFD BDS Commentary C3.4.5 found that the fatigue stress ranges are negligible in cross-frames and diaphragms of bridges with straight girders and little or no skew. Consequently, this Article is not applicable to the design of the routine steel I-girder bridges that are the subject of this Guide.

3.5 PERMANENT LOADS

3.5.1 Dead Loads: *DC*, *DW*, and *EV*

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article defines the permanent dead loads applicable to the design of bridges and the unit weights that may be applied to those loads. DC and DW are applicable to superstructure design. Vertical earth loads, EV, do not apply to the superstructure design of the routine steel I-girder bridges covered by this Guide.

3.5.2 Earth Loads: *EH*, *ES*, and *DD*

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

This Article defines earth loads, which are applicable to the design of substructures, culverts, retaining walls, tunnels, and similar structures, but not to routine steel I-girder bridge superstructures.

3.6 LIVE LOADS

3.6.1 Gravity Loads: *LL* and *PL*

3.6.1.1 Vehicular Live Load

3.6.1.1.1 Number of Design Lanes

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article specifies the method for determining the number of design lanes for applying live load to the structure, which is applicable to all routine steel I-girder bridges. Owners occasionally may specify more conservative guidelines for the determination of the number of design lanes which may supersede the provisions of this Article, however the effects of such guidance are generally more significant for substructure design than superstructure design. For routine steel I-girder design, which is based on line girder analysis (such as NSBA's LRFD Simon line-girder analysis and design program) using the approximate live load distribution factors presented in Article 4.6.2.2, the effect of variations in the number of design lanes generally has no impact on the superstructure design. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

3.6.1.1.2 Multiple Presence of Live Load

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article specifies the multiple presence factors applied to live load effects, which account for the probability of concurrent vehicles in adjacent lanes. These provisions are applicable to the design of bridges carrying vehicular traffic, including the routine steel I-girder bridges covered by this Guide.

Care should be taken in the application of this factor when using the approximate live load distribution factors of Articles 4.6.2.2, as is the case in the line girder analysis methods used to design routine steel I-girder bridges (such as NSBA's LRFD Simon line-girder analysis and design program). Users should verify the assumptions and general correctness of any program's calculations prior to initial use. As noted in the first paragraph of the Commentary, the multiple presence factors are already included in the approximate equations for the live load distribution factors given in the tables in Articles 4.6.2.2 and 4.6.2.3 and should not be applied when using these equations. Also, as noted in the first paragraph of this Article, the multiple presence factors are not to be applied at the fatigue limit state. Therefore, when investigating fatigue using the fatigue design load placed in a single lane using the single-lane distribution factor equations given in Articles 4.6.2.2 and 4.6.2.3, as applicable, the computed distribution factor from the equation must be divided by the specified multiple presence factor for one-lane loaded of 1.2 (Table 3.6.1.1.2-1). When utilizing the lever rule or the special rigid cross-section requirement for evaluating the single-lane live-load distribution at the fatigue limit state to the exterior girder in steel I-girder bridges (see the Discussion of Articles 4.6.2.2.2d and 4.6.2.2.2e in the Guide), the multiple presence factor of 1.2 should not be applied. However, when utilizing the lever rule or the special rigid cross-section requirement for evaluating the live-load distribution to the exterior girder in steel I-girder bridges at the strength and service limit states, the appropriate multiple presence factor specified in Table 3.6.1.1.2-1 must be applied. For further description of the lever rule, see the Commentary for Article 4.6.2.2.1.

Designers should carefully read and fully understand the provisions of this Article and its associated Commentary when determining when and how to apply the multiple presence factors. This is especially the case for routine steel I-girder bridges with sidewalk loading

For examples illustrating the proper application of the multiple presence factors in the computation of the live load distribution factors, consult Section 4.4.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures (2015) and NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

3.6.1.2 Design Vehicular Live Load

3.6.1.2.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article defines the components (truck, tandem and lane) comprising the HL-93 design vehicle, which is applicable to the design of all bridges, including routine steel I-girder bridges.

Note that most commercial line girder analysis steel bridge design programs, such as NSBA's LRFD Simon line-girder analysis and design program, are pre-programmed with the AASHTO LRFD BDS standard live loads as either the default live load or a selectable live load option. Designers should verify their understanding of the program's live load model and how it is used in the analysis prior to initial use of the program but should rarely have to separately program the standard AASHTO LRFD BDS live loads.

Note that some Owners may prescribe that designs use a modified version of these live load components or may specify that designs also consider additional Owner-specific live loads; review Owner-agency guidelines in conjunction with the provisions of this Article.

For further information on live loads, consult section 3.4 and 4.4 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures (2015). The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

3.6.1.2.2 Design Truck

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article defines loading and axle spacing of the HL-93 design truck, which is applicable to all routine steel I-girder bridges and is applied in conjunction with the design lane load (see the Discussion of Articles 3.6.1.2.4 and 3.6.1.3 in this Guide). The dynamic load allowance, IM, of 33 percent specified in Article 3.6.2.1 (see the Discussion of Article 3.6.2.1 in this Guide) is applied to the design truck only.

Note that most commercial line girder analysis steel bridge design programs, such as NSBA's LRFD Simon line-girder analysis and design program, are preprogrammed with the AASHTO LRFD BDS standard live loads as either the default live load or a selectable live load option. Designers should verify their understanding of the program's live load model and how it is used in the analysis prior to initial use of the program but should rarely have to separately program the standard AASHTO LRFD BDS live loads.

Note that some Owners may prescribe that designs use a modified version of these live load components or may specify that designs also consider additional Owner-specific live loads; review Owner-agency guidelines in conjunction with the provisions of this Article.

For further information on live loads, consult section 3.4 and 4.4 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures (2015). The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

3.6.1.2.3 Design Tandem

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article defines loading and axle spacing of the HL-93 design tandem, which is applicable to all routine steel I-girder bridges and is applied in conjunction with the design lane load (see the Discussion of Articles 3.6.1.2.4 and 3.6.1.3 in this Guide). The dynamic load allowance, IM, of 33 percent specified in Article 3.6.2.1 (see the Discussion of Article 3.6.2.1 in this Guide) is applied to the design tandem only. This combination of the design tandem and design lane load will typically only control the live-load force effects for short-span bridges.

Note that most commercial line girder analysis steel bridge design programs, such as NSBA's LRFD Simon line-girder analysis and design program, are preprogrammed with the AASHTO LRFD BDS standard live loads as either the default live load or a selectable live load option. Designers should verify their understanding of the program's live load model and how it is used in the analysis prior to initial use of the program but should rarely have to separately program the standard AASHTO LRFD BDS live loads.

Note that some Owners may prescribe that designs use a modified version of these live load components or may specify that designs also consider additional Owner-specific live loads; review Owner-agency guidelines in conjunction with the provisions of this Article.

For further information on live loads, consult section 3.4 and 4.4 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures (2015). The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

3.6.1.2.4 Design Lane Load

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article defines loading and lane with of the HL-93 lane load, which is applicable to all routine steel I-girder bridges and is applied in conjunction with the design truck or design tandem (see the Discussion of Articles 3.6.1.2.2, 3.6.1.2.3, and 3.6.1.3 in this Guide). The dynamic load allowance, IM, of 33 percent specified in Article 3.6.2.1 (see the Discussion of Article 3.6.2.1 in this Guide) is not applied to the design lane load.

Note that most commercial line girder analysis steel bridge design programs, such as NSBA's LRFD Simon line-girder analysis and design program, are preprogrammed with the AASHTO LRFD BDS standard live loads as either the default live load or a selectable live load option.

Designers should verify their understanding of the program's live load model and how it is used in the analysis prior to initial use of the program but should rarely have to separately program the standard AASHTO LRFD BDS live loads.

Note that some Owners may prescribe that designs use a modified version of these live load components or may specify that designs also consider additional Owner-specific live loads; review Owner-agency guidelines in conjunction with the provisions of this Article.

For further information on live loads, consult section 3.4 and 4.4 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures (2015). The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

3.6.1.2.5 Tire Contact Area

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

This Article defines the tire contact area for the design truck and tandem. The tire contact area is generally considered only in the local design of bridge decks. For the routine steel I-girder bridges covered by this Guide, it is assumed that an owner-specified deck design is being used. For the superstructure (girder) design of routine bridges, idealizing the wheel or axle loads as point loads is the standard accepted method.

3.6.1.2.6 Distribution of Wheel Load through Earth Fills

3.6.1.2.6a General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies the method of distributing live load forces through earth fill for buried structures, which does not apply to the design of the routine steel I-girder bridges covered by this Guide.

3.6.1.2.6b Traffic Parallel to the Culvert Span

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies live load distribution factors for culverts subjected to traffic parallel to the structure, which do not apply to the design of the routine steel I-girder bridges covered by this Guide.

3.6.1.2.6c Traffic Perpendicular to the Culvert Span

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies live load distribution factors for culverts subjected to traffic perpendicular to the structure, which do not apply to the design of the routine steel I-girder bridges covered by this Guide.

3.6.1.3 Application of Design Vehicular Live Loads

3.6.1.3.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article describes the method for applying the design vehicular live loads to the structure. Most, but not all, of the provisions are applicable to the design of the routine steel I-girder bridges covered by this Guide as described further below. Note that axle loads or lengths of the design lane that do not contribute to the extreme force effect under consideration are to be neglected.

The particular item in this Article related to the design of deck overhangs is not applicable to the design of routine steel I-girder bridge superstructures in the context of this Guide, since the design of the deck (and thus the deck overhang) is beyond the scope of this Guide. When calculating the effect of live loads on the steel superstructure, the other provision which states that the center of any wheel load not be closer than 2.0 feet from the edge of the design lane is applicable, particularly when utilizing the lever rule or the special rigid cross-section requirement for evaluating the live-load distribution to the exterior girder in steel I-girder bridges (see the Discussion of Articles 4.6.2.2.2d and 4.6.2.2.2e in this Guide).

For simple span bridges, the particular item concerning the effect of loading by 90% of two design trucks in a single lane with a *minimum* headway of 50 feet (and a rear-axle spacing fixed at 14 feet) combined with 90% of the design lane load for negative moments between points of permanent load contraflexure is not applicable to the design of simple span bridge superstructures, which are subject only to positive moment loading. This item may be applicable to the calculation of reactions at interior piers in bridges with multiple simple spans, but substructure design is beyond the scope of this Guide.

For multi-span continuous bridges, the particular item concerning the effect of loading by 90% of two design trucks in a single lane with a *minimum* headway of 50 feet (and a rear-axle spacing fixed at 14 feet) combined with 90% of the design lane load is applicable when calculating negative moments between points of permanent load contraflexure and reactions at interior supports in routine multi-span continuous rolled beam and plate girder bridges. The negative moments between points of permanent load contraflexure and interior-support reactions due to this loading are compared to the negative moments in these regions and interior-support reactions due to the HL-93 loading and the governing moment and reaction applies. Due to the minimum 50-foot headway between trucks and the 0.90 reduction factor, this loading will generally not control for continuous structures with short spans (i.e., less than 50 feet). The dynamic load allowance, IM, of 33 percent specified in Article 3.6.2.1 (see the Discussion of Article 3.6.2.1 in this Guide) is applied to the two design trucks only.

For further information on live loads, consult section 3.4 and 4.4 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures (2015). The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

3.6.1.3.2 Loading for Optional Live Load Deflection Evaluation

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article defines the special live loading for the optional live load deflection evaluation specified in Article 2.5.2.6.2 (see the Discussion of Article 2.5.2.6.2 in this Guide). This special loading should be used to evaluate the live load deflection in the routine steel I-girder bridges covered by this Guide, unless specified otherwise by the Owner-agency. The dynamic load allowance, IM, of 33 percent specified in Article 3.6.2.1 (see the Discussion of Article 3.6.2.1 in this Guide) is applied to the design truck portion of the loading only.

For further information on live loads, consult section 3.4 and 4.4 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures (2015). The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

3.6.1.3.3 Design Loads for Decks, Deck Systems, and the Top Slabs of Box Culverts

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

This Article describes the live loading of decks and top slabs of culverts. Only the provisions related to deck design are applicable to the routine steel I-girder bridges covered by this Guide; however, deck design is outside the scope of this Guide.

3.6.1.3.4 Deck Overhang Load

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

This Article describes the live loading for the design of deck overhangs, which is outside the scope of this Guide.

3.6.1.4 Fatigue Load

3.6.1.4.1 Magnitude and Configuration

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article provides the axle loads and spacing for the fatigue live load vehicle, which is equivalent to the design truck specified in Article 3.6.1.2.2 but with a constant rear-axle spacing of 30 feet. The fatigue live load vehicle is placed in a single lane and the design lane load is not applied. The dynamic load allowance, IM, of 15 percent specified in Article 3.6.2.1 (see the Discussion of Article 3.6.2.1 in this Guide) is applied to the fatigue live load vehicle. Fatigue loading is applicable to all steel girder bridges carrying vehicular live load.

The third paragraph and associated figure concerning orthotropic decks is not applicable to the routine steel I-girder bridges covered by this Guide, which do not use orthotropic steel decks.

3.6.1.4.2 Frequency

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article defines the average number of trucks per day in a single lane averaged over the design life of the structure, $ADTT_{SL}$. This is used to determine if a particular fatigue detail should be designed for finite or infinite fatigue life in order to calculate the appropriate nominal fatigue resistance for the detail in question (see the Discussion of Article 6.6.1.2.3 in this Guide regarding the use of the $ADTT_{SL}$ in determining whether a fatigue detail is to be designed for finite or infinite life).

The $ADTT_{SL}$ is calculated from Eq. 3.6.1.4.2-1 using the average daily truck traffic *in one direction* averaged over the design life, or *ADTT*. The *ADTT* should be determined in consultation with the traffic engineers to obtain a best estimate of traffic over the life of the structure. In most cases, traffic count data is only available for the “current year” and a “design year,” which is often 20 years into the future. The traffic count data for the future “design year” is generally what should be used as the basis for fatigue analysis calculations; the current year should not be used, and there is no need to try to extrapolate for the 75-year or 100-year service life of the structure. Should a bidirectional *ADTT* be provided, the Commentary for this Article recommends designing for 55 percent of the bidirectional *ADTT* to determine the *ADTT* in one direction. The fraction of truck traffic in a single lane, *p*, determined from Table 3.6.1.4.2-1 should be based on the number of design lanes (see the Discussion of Article 3.6.1.1.1 in this Guide).

If a reasonable estimate of the $ADTT_{SL}$ cannot be made due to lack of traffic data or some other reason, consideration may be given to conservatively designing fatigue details for infinite fatigue life, which does not require the $ADTT_{SL}$. However, in the routine multi-span continuous I-girder bridges covered by this Guide, the Category C' fatigue check of the connection plate-to-bottom flange weld in regions near the points of permanent load contraflexure may control the size of the bottom flange. Therefore, a reasonable estimate of the $ADTT_{SL}$ should be made, if possible, to perform a more accurate assessment of the nominal fatigue resistance for either finite or infinite life, as applicable.

The welded connections typically used to attach angle- or tee-section (WT) cross-frame members to gusset plates in the truss-type cross-frames used in many steel I-girder bridges are identified as Category E' details, and as such have very low fatigue resistance. As a result, to achieve a reasonable cross-frame design, designers often choose to evaluate the cross-frames for finite life (if the $ADTT_{SL}$ is such that finite-life design is permitted for a Category E' detail) even if the girders are being conservatively designed for infinite fatigue life. However, as explained in the Discussion of Article 6.6.1.2.1 in this Guide, designers need not be concerned about performing a fatigue analysis of cross-frame or diaphragm members in routine steel I-girder bridges; due to the nature of the geometry of the framing plan and overall layout of routine steel I-girder bridges, the live load force effects (and the resulting live load stress ranges) in the cross-frames or diaphragms are not significant. Also, cross-frame force effects cannot be directly calculated by a line-girder analysis. Thus, it is reasonable, permitted, and recommended that designers do not try to evaluate the cross-frames or diaphragms for fatigue in the routine steel I-girder bridges covered by this Guide.

3.6.1.4.3 Load Distribution for Fatigue

3.6.1.4.3a Refined Methods

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies the placement of live load for refined methods of analysis, such as 2D grid or grillage analysis, 2D plate-and-eccentric-beam analysis, or 3D analysis. For routine steel I-girder bridges, only line girder analysis (such as the analysis performed by NSBA's LRFD Simon line-girder analysis and design program) is needed and therefore this Article is not applicable. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

3.6.1.4.3b Approximate Methods

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article specifies the distribution factor to be used for a single lane of fatigue live load when using approximate methods of analysis such as line girder analysis; therefore this Article is applicable to the design of the routine steel I-girder bridges covered by this Guide.

3.6.1.5 Rail Transit Load

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article discusses rail transit live loads, which are considered not applicable in the design of routine steel I-girder bridges for the purposes of this Guide.

3.6.1.6 Pedestrian Loads

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article defines pedestrian loads, which are only applicable to highway bridges providing pedestrian access (i.e., sidewalks). The Article also specifies the method for combining pedestrian and vehicular loads to determine the total live load on a girder. Where vehicular traffic can occupy the sidewalk in the as-designed condition or a potential future configuration, a loading condition where the design vehicular live load is assumed to act over the entire clear width of the bridge (including the width of the sidewalk) should be considered (but not acting concurrently with pedestrian load). Depending on sidewalk width, simple hand calculations may demonstrate that this condition (vehicular traffic over full width of the structure) controls over the case of pedestrian loading on the sidewalk plus vehicular loading away from the sidewalk, thereby eliminating the need to analyze the additional loading configurations.

3.6.1.7 Loads on Railings

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

This Article refers to Section 13 for the applicable live load on railings. Railing, or barrier, design is beyond the scope of this Guide.

3.6.2 Dynamic Load Allowance: *IM*

3.6.2.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article provides the dynamic load allowance, IM, factors to apply to the static force effects of the design truck or design tandem to account for dynamic effects as the vehicle crosses the structure. The dynamic load allowance is not to be applied to the force effects due to the design lane load or due to pedestrian loads. The provisions directly related to the design of bridges are applicable to the design of the routine steel I-girder bridges covered by this Guide.

The provisions mentioned in this Article for the application of the dynamic load allowance for buried components and for deck joints are not applicable to the routine steel I-girder bridge superstructures covered by this Guide.

In addition, the provision which allows for the reduction of the dynamic load allowance "if justified by sufficient evidence" generally should not be used in the design of routine steel I-girder bridges without truly sufficient evidence (such as site-specific experimental testing) and the input and approval of the Owner-agency.

3.6.2.2 Buried Components

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides the dynamic load allowance for buried components, which is not applicable to the superstructure design of the routine steel I-girder bridges covered by this Guide.

3.6.2.3 Wood Components

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article discusses the dynamic load allowance for wood components, which is not applicable to the design of the routine steel I-girder bridges covered by this Guide.

3.6.3 Centrifugal Forces: *CE*

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies the magnitude of lateral loading, or centrifugal force, acting on vehicles as they traverse a bridge on a horizontally curved alignment. Routine steel I-girder bridges, as defined for the purposes of this Guide, do not have curved decks and thus presumably would not have curved lanes; in such cases there would not be any centrifugal force effects to consider.

However, even in the rare instance of a bridge with a straight deck but curved lanes, consideration of centrifugal force effects in the design of the superstructure is unlikely to be warranted. The design of routine steel I-girder bridges is, by definition for the purposes of this Guide, based on line girder analysis (such as NSBA's LRFD Simon line-girder analysis and design program). The effects of centrifugal force on the distribution of loads to the left and right wheels of a vehicle cannot be considered in a line girder analysis, in which the distribution of live loads is based on approximate live load distribution factors, not on refined distribution of individual wheel loads. These approximate live load distribution factors are conservative, and it is unlikely that there would be a case of a bridge with a straight deck (part of the definition of a "routine steel I-girder bridge" for the purposes of this Guide) with lanes curved significantly enough for centrifugal force effects to be of concern.

Centrifugal force effects should always be considered in the design of substructures for bridges with curved decks and/or curved lanes, but substructure design is beyond the scope of this Guide.

3.6.4 Braking Force: *BR*

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

This Article defines a longitudinal load acting 6 feet above the deck level to account for the force effects of vehicular braking. The principal effects of this loading are on the bearings, substructures, and foundations. The applied braking loads do not have a measurable influence on the design forces of girders (moment and shear) or cross-frames. It can be shown that any redistribution in vehicular weight to the axle loads due to braking will have a negligible effect on girder design. Therefore, this Article is not applicable to the superstructure design of routine steel I-girder bridges and consideration of the effects of braking forces are beyond the scope of this Guide.

3.6.5 Vehicular Collision Force: *CT*

3.6.5.1 Protection of Structures

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

This Article defines which substructure items are subject to vehicular collision forces, the associated loading conditions, and means of mitigating or avoiding the subjecting of substructure units to crash loads. Vehicular collision loads are typically not a consideration in new structures where site constraints allow either a geometric layout that provides a proper clear zone or an appropriate barrier to protect the substructure units. Regardless, vehicular collision is primarily a substructure design consideration in most bridges, and the definition of a routine steel I-girder bridge for the purposes of this Guide specifically excludes bridges where the superstructure may be subject to vehicular collision forces.

3.6.5.2 Vehicle Collision with Barriers

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

This Article covers the design of rigid barriers used to protect substructure elements, which is beyond the scope of this Guide.

3.7 WATER LOADS: *WA*

3.7.1 Static Pressure

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article discusses the static pressure of water exerted on water-retaining structures, which is not applicable to the superstructure design of the routine steel I-girder bridges covered by this Guide.

3.7.2 Buoyancy

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies buoyancy uplift forces for structures inundated by water. This Article only applies for stream or river crossings when the design flood level is above the bottom flange for the entire length of span, and only when ventilation is not available. The definition of a routine steel I-girder bridge for the purposes of this Guide specifically excludes bridges where there is insufficient freeboard between the superstructure low chord and the high-water elevation associated with design flood events.

3.7.3 Stream Pressure

3.7.3.1 Longitudinal

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

This Article defines the longitudinal stream pressure force acting on substructure units subjected to flowing water. This Article is only applicable to substructure design and is beyond the scope of this Guide. Recall that the definition of a routine steel I-girder bridge for the purposes of this Guide specifically excludes bridges where there is insufficient freeboard between the superstructure low chord and the high-water elevation associated with design flood events.

3.7.3.2 Lateral

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

This Article defines the lateral stream pressure force acting on substructure units subjected to flowing water. This Article is only applicable to substructure design and is beyond the scope of this Guide. Recall that the definition of a routine steel I-girder bridge for the purposes of this Guide specifically excludes bridges where there is insufficient freeboard between the superstructure low chord and the high-water elevation associated with design flood events.

3.7.4 Wave Load

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies that wave action, where applicable, shall be considered. This Article only applies to bridges where wave action is a consideration and would be very rare. Recall that the definition of a routine steel I-girder bridge for the purposes of this Guide specifically excludes bridges where there is insufficient freeboard between the superstructure low chord and the high-water elevation associated with design flood events.

3.7.5 Change in Foundations Due to Limit State for Scour

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

This Article discusses scour and only applies to the substructures of stream or river crossing bridges. New routine steel I-girder bridges are generally designed to avoid scour activity affecting the superstructure; therefore, this Article does not apply.

3.8 WIND LOAD: *WL* AND *WS*

3.8.1 Horizontal Wind Load

3.8.1.1 Exposure Conditions

3.8.1.1.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article defines the superstructure area exposed to wind and the associated attack angles. For the routine steel I-girder bridges covered by this Guide, wind loading is to be considered when calculating force effects and deflections in the girders prior to deck placement, and possibly during deck placement (see the Discussion of Article 3.8.1.2.1 in this Guide), before the top flange is continuously braced by the concrete deck. After the deck is placed, wind loading is to be considered when determining flange lateral bending moments and stresses in the exterior girder bottom flange, as well as forces in the cross-frame members, due to loading on the exterior girder web.

Note that the Commentary for Article 4.6.2.7.1 provides approximate methods for determining these forces due to wind loading after the deck is placed. The Commentary for Article 4.6.2.7.3 discusses the computation of these forces due to wind loading before the deck is placed.

3.8.1.1.2 Wind Speed

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article specifies the design wind speeds to use at the applicable strength and service limit states.

For routine steel I-girder bridges, the values shown in Figure 3.8.1.1.2-1 are generally sufficient as the basis for calculating design wind pressures; a site-specific wind study is rarely, if ever, warranted.

For the determination of wind speeds during construction and their application, see the AASHTO *Guide Specifications for Wind Loads on Bridges During Construction*, and also consider local Owner-agency policies which may address this topic. In addition, the Reference Manual for NHI Course 130102, Engineering for Structural Stability in Bridge Construction provides guidance on selection of design wind speeds during construction. The reader is cautioned that the Reference Manual for NHI Course 130102 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

3.8.1.1.3 Wind Direction for Determining Wind Exposure Category

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article specifies the wind directions for determining the exposure category. The Article states that for "typical bridges, the wind exposure category as specified in Article 3.8.1.1.5 shall be perpendicular to the bridge." Routine steel I-girder bridges generally fall under this designation; consideration of multiple directions is generally unnecessary for these types of structures.

3.8.1.1.4 Ground Surface Roughness Categories

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article defines the ground surface roughness categories used for determining applicable static wind load pressures. The categories match those used in ASCE 7-10, Minimum Design Loads for Buildings and Other Structures. Helpful photographs of typical Category B and C conditions are shown in ASCE 7-10 at the end of Chapter C26, and are also provided in Section 7 of the Reference Manual for NHI Course 130102, Engineering for Structural Stability in Bridge Construction. For a particular structure, if the category isn't clear (e.g., Category B or C), choosing the higher category (i.e., C), will result in higher loads. For the routine steel I-girder bridges covered by this Guide, the wind load often does not dictate the girder dimensions.

3.8.1.1.5 Wind Exposure Categories

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article defines the wind exposure category based on ground surface roughness as determined in Article 3.8.1.1.4 and its prevalence upwind from the structure. Section C26.7 of ASCE 7-10, Minimum Design Loads for Buildings and Other Structures, provides figures that help clarify the definition of conditions upwind from the structure.

3.8.1.2 Wind Load on Structures: WS

3.8.1.2.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article specifies the equation for determining the static design wind pressure acting on the structure and the associated variables. The Design 3-second Gust Wind Speed, V, is taken from Article 3.8.1.1.2 for the various final condition limit states. The equations to determine the pressure exposure and elevation coefficient, K_Z, for the Strength III and Service IV limit states are presented in this Article; however, the commentary for this Article provides predetermined values for the various exposure categories at regular intervals of structure height. For routine steel I-girder bridge superstructure design, any refinement beyond using the maximum superstructure height about the ground or water is generally not warranted. Additionally, for routine steel I-girder bridges without sound barriers, the Gust Effect Factor, G, and Drag Coefficient, C_D, are typically 1.00 and 1.3, respectively.

Designers should also consider including the effects of wind loading on the structure during construction for the constructibility checks of the girders (see Discussion of Article 4.6.2.7.3 and Article 6.10.3.1 and related Articles), including consulting with Owner-agency policy regarding loads and load combinations to be evaluated during construction. For the determination of applicable wind pressure and associated variables during construction, see the *AASHTO Guide Specifications for Wind Loads on Bridges During Construction*, and also consider any local Owner-agency policies on this subject. In addition, the Reference Manual for NHI Course 130102, Engineering for Structural Stability in Bridge Construction provides guidance on the determination of wind pressures during construction. Generally, two levels of wind should be considered, an inactive and active wind. The higher inactive wind load is applied to the fully erected steel structure to check loads and stability prior to deck placement. The lower active wind load is the force to apply during the placement of the noncomposite, or wet, concrete weight, which corresponds to the maximum expected wind speed that would occur during deck placement operations. Often, the magnitude of this active wind load pressure may be found to be very small and may be neglected in the analysis of deck placement loads.

3.8.1.2.2 Loads on the Superstructure

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article discusses the wind load and directions of application for the design of superstructures. For the routine steel I-girder bridges covered by this Guide, specifically for the design of the superstructure, this discussion should be viewed in two contexts: wind loads during construction; and wind loads on the completed structure. During construction, the steel superstructure elements typically investigated for the effects of wind load include the noncomposite girder top and bottom flanges (evaluated as part of the girder constructibility checks), and the cross-frames or diaphragms. In the completed structure, steel superstructure elements typically investigated for the effects of wind load include the girder bottom flanges and the cross-frames or diaphragms. The wind effects in these elements are controlled by wind acting perpendicular to the bridge; other wind skew angles do not need to be investigated.

3.8.1.2.3 Loads on the Substructure

3.8.1.2.3a Loads from the Superstructure

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

This Article discusses the application of wind effects from the superstructure as they are applied to the substructure including the various angles of attack to be considered. In general, this Article is only applicable to substructure design and is beyond the scope of this Guide.

3.8.1.2.3b Loads Applied Directly to the Substructure

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

This Article provides requirements for applying wind loads directly to the substructure elements, which is beyond the scope of this Guide.

3.8.1.2.4 Wind Loads on Sound Barriers

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article only applies to bridges supporting sound barriers. For the purposes of this Guide, the definition of a routine steel I-girder bridge excludes bridges which have sound barriers.

3.8.1.3 Wind Load on Live Load: WL

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article defines the transverse wind acting on live load and acting 6 feet above the deck level. The applied wind on live load does not have a measurable influence on the design forces of girders (moment and shear) or intermediate cross-frames. Wind on live load is primarily a design consideration for bearing and substructure design. However, the transmission of the load from the superstructure (resisted by diaphragm action of the concrete deck) to the bearings though the cross-frames or diaphragms at the supports must be considered in the design of those elements. Similar to wind load acting on the superstructure, wind on live load acting perpendicular to the bridge is generally the controlling direction for the design of cross-frames or diaphragms at the supports. Wind on live load acting at other angles of attack as discussed in this Article is applicable to the design of the bearings and substructure, which is beyond the scope of this Guide.

3.8.2 Vertical Wind Load

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

This Article specifies an upward, or overturning, wind load acting on the superstructure in conjunction with the transverse superstructure wind for the Strength III and Service IV limit states. For the routine steel I-girder bridges covered by this Guide, the effect on the superstructure design (girders and cross-frames) is negligible and can be ignored. The uplift wind must be considered, however, in the design of the bearings and substructure, which is beyond the scope of this Guide.

3.8.3 Wind-Induced Bridge Motions

3.8.3.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article and other sub-Articles of Article 3.8.3 specify requirements for bridges subjected to wind-induced vibrations. The routine steel I-girder bridges covered by this Guide do not have a span-to-depth or length-to-width ratio exceeding 30, cable supports, or, in general, fundamental vertical or translational periods greater than 1 second; therefore, this Article is not applicable.

3.8.3.2 Wind-Induced Motions

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The routine steel I-girder bridges covered by this Guide are not subject to wind-induced vibrations; therefore this Article is not applicable.

3.8.3.3 Control of Dynamic Responses

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The routine steel I-girder bridges covered by this Guide are not subject to wind-induced vibrations; therefore this Article is not applicable.

3.8.4 Site-Specific and Structure-Specific Studies

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The routine steel I-girder bridges covered by this Guide are not subject to wind-induced vibrations; therefore this Article is not applicable.

3.10 EARTHQUAKE EFFECTS: EQ

3.10.2 Seismic Hazard

3.10.2.1 General Procedure

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article presents horizontal peak ground acceleration coefficients which are used to determine the peak ground acceleration coefficient, *PGA*. The PGA is used to calculate the design forces discussed in Article 3.10.9.2 (see the Discussion of Article 3.10.9.2 in this Guide). Other values such as the short- and long-period spectral acceleration coefficients are not pertinent or applicable to the design of the routine steel I-girder bridge superstructures covered by this Guide.

3.10.3 Site Effects

3.10.3.2 Site Factors

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article presents various site factors. The only site factor which is applicable to the design of routine steel I-girder bridge superstructures is the Site Factor F_{pga}. The Site Factor F_{pga} is found in Table 3.10.3.2-1. For Bridges in Seismic Zone 1, always use Site Class B, and identify the value of the Site Factor F_{pga} from the table as a function of the peak ground acceleration coefficient, PGA, determined in Article 3.10.2.1 (see the Discussion of Article 3.10.2.1 in this Guide).

The Site Factor F_{pga} is used to calculate the design forces discussed in Article 3.10.9.2. Other values Site Factors F_a and F_v are not pertinent or applicable to the design of the routine steel I-girder bridges covered by this Guide.

3.10.4 Seismic Hazard Characterization

3.10.4.2 Elastic Seismic Response Coefficient

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article discusses elastic seismic response coefficients. Only bridges in Seismic Zone 1 meet the definition of a routine steel I-girder bridge for the purposes of this Guide, and thus only Article 3.10.9.2 is applicable, and Article 3.10.9.2 only refers to this Article (3.10.4.2) for the purposes of determining the applicable acceleration coefficient, A_s, as specified in Eq. 3.10.4.2-2. The two variables in Eq. 3.10.4.2-2 are the peak ground acceleration coefficient on rock (Site Class B), *PGA*, and the Site Factor F_{pga} which are specified in Articles 3.10.2.1 and 3.10.3.2, respectively.

3.10.9 Calculation of Design Forces

3.10.9.2 Seismic Zone 1

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article specifies horizontal design connection forces between the superstructure and substructure for bridges in Seismic Zone 1. Bridges in Seismic Zone 1 meet the definition of a routine steel I-girder bridge for the purposes of this Guide, so the provisions of this Article are applicable to their design. The provisions for the calculations of the forces are a function of the acceleration coefficient, which is specified in Article 3.10.4.2 (see the Discussion of Article 3.10.4.2 in this Guide).

Note that the design forces discussed this Article are only for the “connection” of the superstructure to the substructure, and the definition of “connections” (as discussed in the Commentary for Article 3.10.7.1) includes only fixed bearings, expansion bearings with either restrainers, shock transmission units (STUs), or dampers, and shear keys. There is no need to apply these forces to the design of pier or end cross-frames or other elements of the superstructure. See the related Discussion of Article 6.16.3 in this Guide.

SECTION 4: STRUCTURAL ANALYSIS AND EVALUATION

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4.4 ACCEPTABLE METHODS OF STRUCURAL ANALYSIS

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article discusses acceptable methods of structural analysis. While many of these methods could be used to analyze routine steel I-girder bridges, only “classical force and displacement methods,” specifically line girder analysis, is necessary. While more refined, and more complex, analysis methods might yield nominally more efficient designs, the benefits rarely justify the increased level of effort for these types of structures. Additionally, most Owners prefer to use line girder analysis methods for load rating routine steel I-girder bridges; they rarely have the time or budget to use refined analysis for load rating the simpler structures in their inventory. “Optimizing” a design through the use of refined analysis methods may result in a situation where the bridge cannot be successfully rated in the future using the Owner’s typical line girder analysis rating methods.

4.5 MATHEMATICAL MODELING

4.5.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article and its associated Commentary discuss a variety of modeling topics. Most of these topics are not applicable to the design of routine steel I-girder bridges and should not be explored for those types of designs.

This Article discusses the consideration of continuous composite barriers as part of the superstructure stiffness in analysis. For the design of routine steel I-girder bridges, relying on inclusion of barrier stiffness to meet deflection, service, and fatigue requirements is neither necessary nor advisable. Inability to meet these requirements using the stiffness of the girders and deck alone typically indicates that the depth, size, and/or spacing of the girders are inadequate.

The Article also mentions that the analysis should recognize the vertical freedom of the girder at bearings where lift-off is indicated. This would necessitate consideration of geometric nonlinearity in the analysis. Line girder analysis programs (such as NSBA's LRFD Simon line-girder analysis and design program) typically cannot address nonlinear behavior, and the use of refined analysis methods which could address nonlinear behavior is not warranted for routine steel I-girder bridges. Instead, routine steel I-girder bridges should be designed such that they are not subject to lift-off at any bearings. If the analysis of a routine steel I-girder bridge suggests there might be uplift/lift-off at any bearing, steps should be taken to eliminate or prevent uplift/lift-off. Typically, if a design is subject to uplift, it will be indicated in a line girder analysis if the net bearing reactions reported by the analysis program are “negative” (i.e., if the reactions are acting to hold down the superstructure at a given bearing). Note that “live load uplift” (i.e., “negative” bearing reactions under the specific load case of live load alone) may not be a problem, as long as the uplift under live load is less than the downward bearing reaction due to dead load (with appropriate load factors applied to all load cases to determine the overall net reaction for the load combination).

Uplift at bearings (under either service level or factored loading) should not be permitted in a routine steel I-girder bridge, and in fact most Owner-agencies have policies (either explicit, written policies or implicit, unwritten policies) prohibiting uplift at bearings. If the line girder analysis results are showing “negative” net reactions, steps should be taken to eliminate the uplift. The best solution for addressing uplift in a routine steel I-girder bridge is to revise the span arrangement / span lengths in such a way that net uplift does not occur. If this is not possible due to site constraints, and uplift is unavoidable, Article 14.6.1 requires that bearings subject to net uplift at any limit state be secured by tie-downs or anchorages, which are generally very complex, costly, and maintenance-prone devices, and which complicate the behavior of the structure. Alternately, counterweights could be provided to eliminate the uplift, but in most cases only relatively small counterweights are actually practical.

The Commentary for this Article discusses the use of elastic and inelastic analysis assumptions. Routine steel I-girder bridges should be analyzed as fully elastic for all limit states, per the definition of these structures for the purposes of this Guide. Inelastic redistribution of negative bending moments introduces levels of complexity and effort which are not warranted for routine steel I-girder bridges.

The other items discussed in this Article are not applicable to the superstructure design of routine steel I-girder bridges.

4.5.2 Structural Material Behavior

4.5.2.1 Elastic Versus Inelastic Behavior

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article defines the basic assumption of elastic-inelastic behavior of structural materials for analysis. Inelastic behavior should not be considered for routine steel I-girder bridges.

For straight continuous-span steel I-girder bridges, the optional provisions of Appendix B6 of the AASHTO LRFD BDS provide rational approaches for calculating moment redistribution from interior-pier sections due to the effects of yielding (see the Discussion of Article 4.6.4.1 in this Guide). This can potentially produce more economical designs, but at the cost of additional analysis and design effort. These approaches utilize elastic moment envelopes, and do not require the direct use of any inelastic analysis methods, but the associated analysis and design considerations are unfamiliar to most designers and commercial software packages do not currently include the capability to automate the associated calculations. As a result, designs which rely on moment redistribution to satisfy AASHTO design criteria will also be more difficult for the Owner-agency to load rate in the future. Therefore, most Owner-agencies currently discourage or prohibit the use of moment redistribution methods for continuous-span steel I-girder bridges.

Additionally, routine steel I-girder bridges, as defined for the purposes of this Guide, do not generally need to recognize inelastic behavior in determining the resistance to extreme event loadings (e.g., seismic loadings).

4.5.2.2 Elastic Behavior

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article discusses elastic behavior as well as stiffness properties of concrete.

The Article suggests that changes in material properties due to concrete maturity should be included in analysis models, where appropriate. This statement warrants discussion and clarification. In situations where the deck is placed in stages, specified by a deck-placement sequence, the stiffness of previously placed and cured sections of the deck should be considered in the analysis, particularly with regard to dead load deflections used to develop the camber diagrams and web-cutting ordinates for the girders, and especially in cases of irregular span balance. Most Owner-agency policies and construction specifications require that previously placed sections of the deck achieve a specified minimum age, minimum strength, or other performance criteria before placing the next section when a staged deck-placement sequence is used; it is generally reasonable in that circumstance to assume the previously placed sections of the deck are close enough to “fully-hardened” to assume they have achieved full stiffness. However, some Owner-agencies’ design policies include consideration of a reduced modulus for previously placed portions of the deck when a staged deck-placement sequence is used. See the Discussion of Article 6.10.3.4.1 in this Guide for additional guidance on this topic.

Conversely, it is neither necessary nor recommended to address changes to material properties during the curing cycle of the deck, as these changes have negligible effects on the behavior or performance of the routine steel I-girder bridges covered in this Guide.

Also, when considering the appropriate level of refinement for the evaluation of dead load deflections, designers are encouraged to keep in mind the typical construction tolerances associated with steel girder fabrication, and the fact that the haunch between the deck and the girder top flange serves to provide geometric adjustability in the field. Trying to quantify deflections to an accuracy of a small fraction of an inch does not provide significant value.

The second paragraph of the Article states that the “stiffness characteristics of beam-slab-type bridges may be based on full participation of concrete decks.” Most line girder analysis programs (such as NSBA's LRFD Simon line-girder analysis and design program) assume the full, uncracked concrete deck for computing stiffness along the girder, including in negative moment regions of continuous span structures. This assumption is reasonable, is standard practice, and is supported by the discussion in the Commentary for Article 6.10.1.5 (refer also to the Discussion of Article 6.10.1.5 in this Guide). Users should verify the capabilities, assumptions, and general correctness of any program’s calculations prior to initial use.

4.5.2.3 Inelastic Behavior

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article discusses inelastic behavior and analysis methods. Inelastic behavior should not be considered for routine steel I-girder bridges.

For straight continuous-span steel I-girder bridges, the optional provisions of Appendix B6 of the AASHTO LRFD BDS provide rational approaches for calculating moment redistribution from interior-pier sections due to the effects of yielding (see the Discussion of Articles 4.6.4.1 and Appendix B6 in this Guide). This can potentially produce more economical designs, but at the cost of additional analysis and design effort. These approaches utilize elastic moment envelopes, and do not require the direct use of any inelastic analysis methods, but the associated analysis and design considerations are unfamiliar to most designers and commercial software packages do not currently include the capability to automate the associated calculations. As a result, designs which rely on moment redistribution to satisfy AASHTO design criteria will also be more difficult for the Owner-agency to load rate in the future. Therefore, most Owner-agencies currently discourage or prohibit the use of moment redistribution methods for continuous-span steel I-girder bridges.

4.5.3 Geometry

4.5.3.1 Small Deflection Theory

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article discusses the limits for the consideration of small deflection theory (i.e., no consideration of second-order effects). Line girder analysis is based on small deflection theory; the moments and shears determined from the elastic analysis are not increased for any second-order effects. Cross-frame axial wind loads calculated by hand are also not increased for second-order effects; however, if the cross-frame members are connected eccentrically, second-order effects must be considered for the resulting moments as detailed in the Discussion of Article 4.5.3.2.1 in this Guide.

4.5.3.2 Large Deflection Theory

4.5.3.2.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article discusses large deflection theory, which is generally not applicable to the superstructure design of routine steel I-girder bridge. One exception is the design of eccentrically connected cross-frame members in compression. Eccentrically connected members are subjected to bending moments when axially loaded. For single-angle members, the resistance equations of Article 6.9.4.4 directly account for these secondary forces and no further action is required. However, for other members, such as WT shapes in compression connected only through their flanges, the designer must account for the increase in moment due to secondary effects when checking the moment-axial force interaction (a.k.a., “beam-column interaction”) per Article

6.9.2.2. These secondary moments may be approximated based on the provisions in Articles 4.5.3.2.2a and 4.5.3.2.2b.

Substructure units of routine steel I-girder bridges may be subjected to second-order effects as discussed in this Article (for example, P-delta moment magnification effects in tall piers); however, substructure design is beyond the scope of this Guide.

4.5.3.2.2 Approximate Methods

4.5.3.2.2a General

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article simply identifies that the use of approximate methods for evaluating the effects of deflections on beam-columns and arches is acceptable. The Commentary for this Article mentions an alternate method, which is generally considered inappropriate and unnecessary for use in routine steel I-girder bridges and provides comments about limitations on actual movements which apply to substructure design, not superstructure design.

4.5.3.2.2b Moment Magnification – Beam Columns

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article presents equations associated with an approximate method for moment magnification of beam columns. This is particularly useful in the design of tall, slender substructure elements (columns or piles) but that application is beyond the scope of this Guide. For the design of routine steel I-girder bridges, these provisions are typically most applicable for the evaluation of tee sections (WT members) with eccentric connections, such as those which may be used in truss-type cross-frames.

4.5.3.2.2c Moment Magnification – Arches

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article specifically apply to the evaluation of arches and are not applicable to the design of routine steel I-girder bridges.

4.5.3.2.3 Refined Methods

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides general requirements for refined methods of analysis which may be used to evaluate structures subject to second-order large deflection theory. The use of these types of refined methods of analysis for the design of routine steel I-girder bridges is neither necessary nor recommended.

A routine steel I-girder bridge, as defined for the purposes of this Guide, is configured in such a manner that refined methods of analysis are not required for its design; instead, routine steel I-girder bridges can, and should, be designed using approximate methods of analysis, specifically line girder analysis methods. As a result, the provisions of this Article do not apply to the design of routine steel I-girder bridges.

4.5.4 Modeling Boundary Conditions

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article requires that boundary conditions represent the actual characteristics of support and continuity. For routine steel I-girder bridges, analyzed using line girder analysis methods, simple boundary conditions are assumed. Line girder analysis programs are one-dimensional analysis methods; the boundary conditions are, by definition, located at the neutral axis of the beam. Line girder analysis also only considers vertical (gravity) loading effects (dead load and live load). Consequently, the boundary conditions for any line girder analysis are limited to the following:

- the vertical translation degree of freedom (DOF) is fixed at all supports;
- the longitudinal translation DOF is fixed at one support (which support does not matter), in order to provide stability for the model;
- all other DOFs (transverse translation and all rotations) are free.

An actual routine steel I-girder bridge, obviously, is a three-dimensional construct, and the bearings are typically located under the bottom flanges of the girders. In the actual structure, the choice of which bearings are fixed against horizontal translation affects the performance of the structure and influences the distribution of horizontal loads among the various bearings and substructures.

Designers should not overthink the boundary conditions in a line girder analysis. But they should think very carefully about how to establish the bearing articulation in the plans.

The AASHTO-NSBA Steel Bridge Collaboration Guideline G13.1-2019 Guidelines for Steel Girder Bridge Analysis provides a discussion of boundary conditions in Section 3.14.

4.5.5 Equivalent Members

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article discusses the methods for modeling nonprismatic (varying) members. For constant-depth girders, such as those used in routine steel I-girder bridges as defined for the purposes of this Guide, most commercial line girder analysis methods (such as NSBA's LRFD Simon line-girder analysis and design program) use appropriate means for discretizing the girder into segments, accounting for changes in flange sizes, etc., such that the requirements of this provision are met; users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use. Performing sensitivity studies of the degree of discretization for specific bridge designs is not necessary, warranted, or recommended.

4.6 STATIC ANALYSIS

4.6.1 Influence of Plan Geometry

4.6.1.1 Plan Aspect Ratio

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article discusses the approximate modeling methods for torsionally-stiff closed cross-sections, such as steel or concrete box-girders. I-girders are open cross-sections, and straight I-girders like those used in routine steel I-girder bridges are quite flexible torsionally.

4.6.1.2 Structures Curved in Plan

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article and its associated sub-Articles discuss the approximate modeling methods for structures curved in plan. For the purposes of this Guide, the definition of routine steel I-girder bridges has been limited to structures which are straight in plan.

4.6.2 Approximate Methods of Analysis

4.6.2.1 Decks

4.6.2.1.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

This Article and its associated sub-Articles address the design of bridge decks. The design of concrete decks for steel I-girder bridges is typically governed by Owner-agency policy manuals (e.g., standard designs, pre-calculated design tables, etc.), and so is not addressed herein.

4.6.2.2 Beam-Slab Bridges

4.6.2.2.1 Application

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article provides the general bounding criteria for use of the AASHTO LRFD BDS approximate live load distribution factors for moment and shear for various types of bridges. Routine steel I-girder bridges as defined for the purposes of this Guide meet these general bounding criteria.

The longitudinal stiffness parameter, K_g, (Eq. 4.6.2.2.1-1) and the appropriate value of the span length, L, (Table 4.6.2.2.1-2) will need to be determined for use in Article 4.6.2.2.2. Routine steel I-girder bridges fall under Table 4.6.2.2.1-1 Case (a).

When line-girder analyses are performed, proper distribution of the loads to the individual girders becomes important to establish the total demand moments in the interior and exterior girders. Since the routine steel I-girder bridges as defined for the purposes of this Guide meet the general bounding criteria given in this Article, the 11th paragraph of this Article permits the DC loads applied to the noncomposite section (referred to herein as DC_1 loads) to be distributed equally to all of the girders of the cross-section for the line-girder analysis. The DC_1 loads consist primarily of the self-weight of the steel, the weight of any stay-in-place forms, and the weight of the wet concrete in the deck. These loads should be assigned equally between all girders in the cross-section if the girders are of approximately equal stiffness at the cross-frame connection points, which is the case for the routine steel I-girder bridges defined for the purposes of this Guide. The intermediate cross-frames or cross-frames act to equalize the girder deflections within a cross-section and nearly equalize the load in equal-stiffness noncomposite girders regardless of the amount of load applied to the individual girders. Using this assumption in these cases, in lieu of the more traditional tributary area assumption applied to the weight of the wet deck concrete and forms, is particularly important in helping to determine more accurate noncomposite deflections, which are used in establishing girder cambers.

To better simulate the actual distribution of the barrier loads, or DC loads applied to the composite section (referred to herein as DC_2 loads) when line-girder analyses are performed, consider assigning a larger percentage of these loads to the exterior girders and the adjacent interior girders, which is a better assumption than a uniform distribution of these loads to all the girders based on an examination of refined analysis results for several cases. Some Owner-agencies have a specific design policy on the distribution of barrier loads; for example, one Owner-agency recommends, for bridges over 44′ in width, distributing the barrier load to the first three exterior girders with 44% of the load applied to the exterior girder, 33% applied to the first interior girder, and 23% applied to the second interior girder.

For the wearing surface load, DW, an equal distribution of the load to all the girders is a reasonable assumption and has been the customary practice.

4.6.2.2.2 Distribution Factor Method for Moment and Shear

4.6.2.2.2a Interior Beams with Wood Decks

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Wood decks are not included in the definition of routine steel I-girder bridges for the purposes of this Guide.

4.6.2.2.2b Interior Beams with Concrete Decks

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article addressed the live load distribution factor for moment on interior beams. The equations in the third row of Table 4.6.2.2.2b-1 ("Concrete Deck or Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-beams, T- and Double T-sections") are the only equations applicable for calculation of live load distribution factors for moment on interior beams of routine steel I-girder bridges. The routine steel I-girder bridges covered by this Guide satisfy the limitations specified in the table for the use of these distribution factors.

Note that the live load distribution factor equations of Table 4.6.2.2.2b-1 inherently include consideration of multiple presence (Article 3.6.1.1.2) as discussed in this Article, in Article 3.6.1.1.2, and associated Commentary for both articles (see also the Discussion of Article 3.6.1.1.2 in this Guide). When evaluating the live load distribution for interior girders at the strength and service limit states, the live load distribution factors calculated from the formulas given in the table should not be modified to account for multiple presence. However, as discussed in the Commentary for Article 3.6.1.1.2 and in the Discussion of Article 3.6.1.1.2 in this Guide, the multiple presence factor of 1.20 should be removed from the one-lane-loaded live load distribution factor for interior girders calculated from the formula given in the table for evaluation of the fatigue limit state.

Section 4.4.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures provides an extensive and helpful discussion of the AASHTO LRFD BDS approximate live load distribution factors for moment in interior girders, including example calculations. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

Most commercial line girder analysis programs (such as NSBA's LRFD Simon line-girder analysis and design program) automatically calculate the live load distribution factors necessary for the analysis. Users should verify the capabilities, assumptions, and general correctness of any program's calculations of the live load distribution factors prior to initial use.

4.6.2.2.2c Interior Beams with Corrugated Steel Decks

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Corrugated steel decks are not included in the definition of routine steel I-girder bridges for the purposes of this Guide.

4.6.2.2.2d Exterior Beams

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article addressed the live load distribution factor for moment on exterior beams. The equations the third row of Table 4.6.2.2.2d-1 ("Concrete Deck or Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-beams, T- and Double T-sections") are the only equations applicable for calculation of live load distribution factors for moment on exterior beams of routine steel I-girder bridges. The routine steel I-girder bridges covered by this Guide satisfy the limitations specified in the table for the use of these distribution factors.

Note that the live load distribution factor equations of Table 4.6.2.2.2d-1 inherently include consideration of multiple presence (Article 3.6.1.1.2) as discussed in this Article, in Article 3.6.1.1.2, and associated Commentary for both articles (see also the Discussion of Article 3.6.1.1.2 in this Guide). When evaluating live load distribution for exterior girders at the strength and service limit states, the live load distribution factor calculated from the formula given in the table for the case of two or more lanes loaded should not be modified to account for multiple presence.

For situations where only one design lane is loaded, the lever rule is used to calculate the distribution factor for moment in an exterior girder. For further description of the lever rule, see the Commentary for Article 4.6.2.2.1. The provisions of Article 3.6.1.1.1 regarding the placement of the design lanes and the placement of the wheel loads within those lanes should be followed when utilizing the lever rule. When evaluating the live load distribution for exterior girders for one-lane loaded at the fatigue limit state utilizing the lever rule, the multiple presence factor of 1.2 should not be applied. When evaluating the live load distribution for exterior girders utilizing the lever rule for situations where only one design lane is loaded at the strength and service limit states, the appropriate multiple presence factor specified in Table 3.6.1.1.2-1 must be applied. The presence or absence of cross-frames or diaphragms is not considered when calculating distribution factors using the lever rule, which only considers the deck acting as a lever supported by the exterior and first interior girder.

In addition, this Article specifies that for steel bridge cross-sections with cross-frames or diaphragms, the live load distribution factor for the exterior girder *is not to be taken less than* that which would be obtained by assuming the cross-section deflects and rotates as a rigid cross-section. This special rigid cross-section analysis is specified because the empirical distribution factors for moment given in the specification table were determined without consideration of cross-frames or diaphragms; hence, while they are conservative for interior girders, they are generally unconservative for exterior girders in steel multi-girder bridges. Therefore, the distribution factor for moment in the exterior girders determined from this special rigid cross-section analysis will usually control and should always be employed for routine steel I-girder bridges since the exterior girder is typically the critical girder for moment. It is recommended that Eq. C4.6.2.2.2d-1 be used to satisfy this assumption; the equation should be evaluated for one lane loaded and also for two or more lanes loaded (up to the total number of design lanes the design roadway width can accommodate). The provisions of Article 3.6.1.1.1 regarding the placement of the design lanes and the placement of the wheel loads within those lanes should also be followed. When evaluating the live load distribution for exterior girders for one-lane loaded at the fatigue limit state utilizing the special analysis, the multiple presence factor of 1.2 should not be applied. When evaluating the live load distribution for exterior girders for any number of design lanes loaded at the strength and service limit states utilizing the special analysis, the appropriate multiple presence factor specified in Table 3.6.1.1.2-1 must be applied.

Section 4.4.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures provides an extensive and helpful discussion of the AASHTO LRFD BDS approximate live load distribution factors for moment in exterior girders, including example calculations utilizing the specification formulas, the lever rule, and the special rigid cross-section analysis. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

Most commercial line girder analysis programs (such as NSBA's LRFD Simon line-girder analysis and design program) automatically calculate the live load distribution factors necessary for the analysis. Users should verify the capabilities, assumptions, and general correctness of any program's calculations of the live load distribution factors prior to initial use.

4.6.2.2.2e Skewed Bridges

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides factors which may be used to reduce the live load distribution factor for moment in the design of skewed bridges. The use of these particular factors is typically at the discretion of the Owner-agency. However, the range of applicability for these provisions is limited to bridges with support skew angles between 30 and 60 degrees. Since the routine steel I-girder bridges as defined for the purposes of this Guide are limited to support skew angles of 20 degrees or less, the reduction factors discussed in this Article cannot be used regardless of the Owner-agency policy.

4.6.2.2.2f Flexural Moments and Shear in Transverse Floorbeams

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Routine steel I-girder bridges as defined for this Guide do not have transverse floorbeams.

4.6.2.2.3 Distribution Factor Method for Shear

4.6.2.2.3a Interior Beams

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article addressed the live load distribution factor for shear on interior beams. The equations in the third row of Table 4.6.2.2.3a-1 ("Concrete Deck or Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-beams, T- and Double T-sections") are the only equations applicable for calculation of live load distribution factors for shear on interior beams of routine steel I-girder bridges. The routine steel I-girder bridges covered by this Guide satisfy the limitations specified in the table for the use of these distribution factors.

Note that the live load distribution factor equations of Table 4.6.2.2.3a-1 inherently include consideration of multiple presence (Article 3.6.1.1.2) as discussed in this Article, in Article 3.6.1.1.2, and associated Commentary for both articles (see also the Discussion of Article 3.6.1.1.2 in this Guide). When evaluating the live load distribution for interior girders at the strength and service limit states, the live load distribution factors calculated from the formulas given in the table should not be modified to account for multiple presence. However, as discussed in the Commentary for Article 3.6.1.1.2 and in the Discussion of Article 3.6.1.1.2 in this Guide, the multiple presence factor of 1.20 should be removed from the one-lane-loaded live load distribution factor for interior girders calculated from the formula given in the table for evaluation of the special fatigue requirement for webs specified in Article 6.10.5.3 (see the Discussion of Article 6.10.5.3 in this Guide). Similarly, the multiple presence factor of 1.20 should also be removed for the determination of the longitudinal shear range for the fatigue design of shear connectors (see the Discussion of Article 6.10.10.1.2 in this Guide). Note that the interior girder is typically the critical girder for shear in a routine steel I-girder bridge.

Section 4.4.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures provides an extensive and helpful discussion of the AASHTO LRFD BDS approximate live load distribution factors for shear in interior girders, including example calculations. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

Most commercial line girder analysis programs (such as NSBA's LRFD Simon line-girder analysis and design program) automatically calculate the live load distribution factors necessary for the analysis. Users should verify the capabilities, assumptions, and general correctness of any program's calculations of the live load distribution factors prior to initial use.

4.6.2.2.3b Exterior Beams

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article addressed the live load distribution factor for shear on exterior beams. The equations in the third row of Table 4.6.2.2.3b-1 ("Concrete Deck or Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-beams, T- and Double T-sections") are the only equations applicable for calculation of live load distribution factors for shear on exterior beams of routine steel I-girder bridges. The routine steel I-girder bridges covered by this Guide satisfy the limitations specified in the table for the use of these distribution factors.

Note that the live load distribution factor equations of Table 4.6.2.2.3b-1 inherently include consideration of multiple presence (Article 3.6.1.1.2) as discussed in this Article, in Article 3.6.1.1.2, and associated Commentary for both articles (see also the Discussion of Article 3.6.1.1.2 in this Guide). When evaluating the live load distribution for exterior girders at the strength and service limit states, the live load distribution factor calculated from the formula for the case of two or more lanes loaded should not be modified to account for multiple presence.

For situations where only one design lane is loaded, the lever rule is used to calculate the distribution factor. For further description of the lever rule, see the Commentary for Article 4.6.2.2.1. The provisions of Article 3.6.1.1.1 regarding the placement of the design lanes and the placement of the wheel loads within those lanes should be followed when utilizing the lever rule. When utilizing the lever rule to determine the live load distribution for exterior girders for one-lane loaded for evaluation of the special fatigue requirement for webs specified in Article 6.10.5.3 (see the Discussion of Article 6.10.5.3 in this Guide) and for the determination of the longitudinal shear range for the fatigue design of shear connectors (see the Discussion of Article 6.10.10.1.2 in this Guide), the multiple presence factor of 1.2 should not be applied. When utilizing the lever rule to determine the live load distribution for exterior girders for situations where only one design lane is loaded at the strength and service limit states, the appropriate multiple presence factor specified in Table 3.6.1.1.2-1 must be applied. The presence or absence of cross-frames or diaphragms is not considered when calculating distribution factors using the lever rule, which only considers the deck acting as a lever supported by the exterior and first interior girder.

In addition, this Article specifies that for steel bridge cross-sections with cross-frames or diaphragms, the live load distribution factor for the exterior girder *is not to be taken less than* that which would be obtained by assuming the cross-section deflects and rotates as a rigid cross-section. It is recommended that Eq. C4.6.2.2.2d-1 be used to satisfy this assumption; the equation should be evaluated for one lane loaded and also for two or more lanes loaded (up to the total number of design lanes the design roadway width can accommodate). The provisions of Article 3.6.1.1.1 regarding the placement of the design lanes and the placement of the wheel loads within those lanes should also be followed. When utilizing the special rigid cross-section analysis to determine the live load distribution for exterior girders for one-lane loaded for evaluation of the special fatigue requirement for webs specified in Article 6.10.5.3 (see the Discussion of Article 6.10.5.3 in this Guide) and for the determination of the longitudinal shear range for the fatigue design of shear connectors (see the Discussion of Article 6.10.10.1.2 in this Guide), the multiple presence factor of 1.2 should not be applied. When utilizing the special analysis to evaluate the live load distribution for exterior girders for any number of design lanes loaded at the strength and service limit states, the appropriate multiple presence factor specified in Table 3.6.1.1.2-1 must be applied. Note that the special rigid-cross section analysis typically does not control for shear.

Section 4.4.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures provides an extensive and helpful discussion of the AASHTO LRFD BDS approximate live load distribution factors for shear in exterior girders, including example calculations utilizing the specification formulas, the lever rule, and the special rigid cross-section analysis. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

Most commercial line girder analysis programs (such as NSBA's LRFD Simon line-girder analysis and design program) automatically calculate the live load distribution factors necessary for the analysis. Users should verify the capabilities, assumptions, and general correctness of any program's calculations of the live load distribution factors prior to initial use.

4.6.2.2.3c Skewed Bridges

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article contains the equations for the correction factor for live load distribution factors for shear in girders at and adjacent to the obtuse corners of skewed supports; the factor is to be applied at all skewed supports (i.e., at both end and interior skewed supports). The equations the first row of Table 4.6.2.2.3c-1 ("Concrete Deck or Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-beams, T- and Double T-sections") are the only equations applicable for calculation of correction factors for routine steel I-girder bridges. These correction factors apply only to the shear distribution factors for the exterior and first interior girder at and adjacent to the obtuse corner of the end and/or interior support and decrease linearly to 1.0 at midspan. The routine steel I-girder bridges covered by this Guide satisfy the limitations specified in the table for the use of the correction factor, and the correction factor is applicable to routine steel I-girder bridges with skewed supports. Be aware that a correction factor is not provided for the dead load shears at skewed supports.

Section 4.4.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures provides an extensive and helpful discussion of the AASHTO LRFD BDS approximate live load distribution factors, including example calculations. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

Most commercial line girder analysis programs (such as NSBA's LRFD Simon line-girder analysis and design program) automatically calculate the live load distribution factors necessary for the analysis. Users should verify the capabilities, assumptions, and general correctness of any program's calculations of the live load distribution factors prior to initial use.

4.6.2.2.4 Curved Steel Bridges

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

By the definition used in this Guide, routine steel I-girder bridges are not curved.

4.6.2.2.5 Special Loads with Other Traffic

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article only applies when the Owner-agency does not have an overriding policy on how to address situations where one lane is loaded with overweight or permit vehicles mixed with routine traffic in the other lanes. The live load distribution equation in this Article is to be used only for cases involving two or more design lanes and is not to be used when use of the lever rule or the rigid cross-section assumption is required by the related Articles for the calculation of live load distribution factors since both of those methods could potentially be used to compute the distribution factor directly.

Section 4.4.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures provides an extensive and helpful discussion of the AASHTO LRFD BDS approximate live load distribution factors, including example calculations. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

Commercial line girder analysis programs may not have the built-in capability to address the case of special loads combined with routine traffic. Some programs may allow the user to overwrite the program's calculated live load distribution factors and substitute factors calculated by the user outside of the program. Users should verify the capabilities, assumptions, and general correctness of any program's calculations of the live load distribution factors prior to initial use.

4.6.2.3 Equivalent Strip Widths for Slab-Type Bridges

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article only applies to slab-type bridges, which are bridges where the main spanning element is a concrete or wood slab, without supporting girders, beams or stringers. Routine steel I-girder bridges are categorized as "beam-slab bridges" in the AASHTO LRFD BDS.

4.6.2.4 Truss and Arch Bridges

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article only applies to truss and arch bridges. Routine steel I-girder bridges are categorized as "beam-slab bridges" in the AASHTO LRFD BDS.

4.6.2.5 Effective Length Factor, K

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article provides guidance on the effective length factor, K, based on various end conditions for the design of compression members. For the design of superstructures for routine steel I-girder bridges, the provisions in this Article apply primarily to the design of truss-type cross-frame members; the appropriate approximate values given in the bulleted items in this Article are typically used. The provisions of this article do not apply to the design of the girders.

The provisions of this Article also apply to a variety of other structural elements which may be present in a bridge, such as columns or piles, but the design of substructure and foundation elements is beyond the scope of this Guide.

4.6.2.6 Effective Flange Width

4.6.2.6.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

The provisions in this Article are used to determine the "effective flange width" of the concrete deck. The effective flange width is used for computing the composite section properties for the composite girder cross-section for determining the composite cross-section stiffness for the analysis and for determining the flexural resistance of the composite section (see the Discussion of Article 6.10.1.1 and the associated sub-Articles in this Guide for further information on the computation of composite section properties).

In composite steel girders subject to major-axis bending, longitudinal stresses are distributed to the various components of the cross-section, including the concrete deck, by in-plane shear stresses resulting in shear deformations. As a result of the corresponding shear deformations in the deck – which is wider and less efficient than the steel girder in distributing the stresses -- plane sections do not remain plane and the longitudinal stresses across the deck are non-uniform; a phenomenon referred to as shear lag. The effective flange width is the width of deck over which the assumed uniformly distributed longitudinal stresses result in approximately the same deck force and member moments calculated from elementary beam theory (i.e. assuming plane sections remain plane) as would be produced by the actual non-uniform stress distribution. As described in the first paragraph of this Article, for the routine steel I-girder bridges covered by this Guide, the effective flange width of the concrete deck should be taken as the corresponding tributary width of the deck perpendicular to the axis of the member. Provisions related to other types of systems mentioned in the remainder of this Article are not applicable. The provisions which allow for extending the deck overhang width used for the analysis to account for the presence of a continuous concrete barrier rail should not be used for routine steel I-girder bridge design. If design requirements cannot be met using the section properties and associated strength of the girder and deck alone, this typically indicates that the depth, size, and/or spacing of the girders are inadequate.

4.6.2.6.2 Segmental Concrete Box Beams and Single-Cell, Cast-in-Place Box Beams

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

These provisions do not apply to the design of steel I-girder bridges.

4.6.2.6.3 Cast-in-Place Multicell Superstructures

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

These provisions do not apply to the design of steel I-girder bridges.

4.6.2.6.4 Orthotropic Steel Decks

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

These provisions do not apply to the design of the routine steel I-girder bridges covered by this Guide, which are assumed not to have orthotropic steel decks.

4.6.2.6.5 Transverse Floorbeams and Integral Bent Caps

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

These provisions do not apply to the design of the routine steel I-girder bridges covered by this Guide, which are assumed not to have transverse floorbeams or integral bent caps.

4.6.2.7 Lateral Wind Load Distribution in Girder System Bridges

4.6.2.7.1 I-Sections

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article addressed the loading of wind perpendicular to the span of the superstructure. Routine steel I-girder bridges, in their final fully constructed condition, have a concrete deck that acts as a lateral diaphragm to transmit lateral wind loads from the top half of the girder directly through the deck. The lateral wind loads on the bottom half of the girder are resisted by lateral bending in the bottom flange and transmitted up to the deck by the cross frames or diaphragms. At the support locations, the transverse wind loads are transmitted from the deck to the support through the cross-frames or diaphragms at those locations.

The Commentary for this Article provides a simplified procedure for calculating the various horizontal loading effects associated with transmitting wind loads through the load paths discussed above. The equations in the procedure are derived from classical equations for moments and reactions in beams, occasionally with modified factors to account for some degree of continuity in the girder flange acting as a beam supported at multiple points by the cross-frames.

Sections 3.5 and 6.5.6.5.1 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures provides an extensive and helpful discussion of the evaluation of the effects of wind loading on the superstructure of steel I-girder bridges, including explanation and background of the provisions of this Article and example calculations. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

4.6.2.7.2 Box Sections

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

These provisions do not apply to the design of steel I-girder bridges.

4.6.2.7.3 Construction

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

During construction, the absence of the hardened, composite concrete deck means that routine steel I-girder bridges behave as structures lacking a deck to provide the horizontal diaphragm action discussed in Article 4.6.2.7.1. The response of these structures to wind loads during construction before the deck placement is completed is significantly different from that of the completed bridge. The structure is much more flexible and subject to larger horizontal deflections under wind loading. Furthermore, the flow of wind around the structure and the resulting wind pressure acting on the individual girders is different.

Another significant difference between bridges during construction and bridges in service is the short length of time expected between the erection of the girders and the placement of the deck. For the same probability of exceedance, the design wind speed decreases with the decrease in the time between the girder erection and the deck placement.

Consequently, the design of routine steel I-girder bridges should include constructibility checks of the steel superstructure in the non-composite condition to resist lateral wind loads, in conjunction with other construction loads, as applicable.

The Commentary to this Article (C4.6.2.7.3) suggests the use of the *AASHTO Guide Specifications for Wind Loads on Bridges During Construction*, which modifies the wind-load provisions discussed in the preceding articles accordingly to account for the differences in the behavior between completed bridges and bridges during construction, to perform these checks. To determine if any wind bracing is necessary, the Guide Specifications may be used to perform an investigation of the inactive work zone wind load case between the completed erection of the girders and the placement of the concrete deck assuming no wind bracing is provided in the plane of either flange. These Specifications may also be used to perform an investigation of the active work zone wind load case during the placement of the deck, if desired. The Commentary goes on to discuss the proportioning of the total calculated lateral wind moment or sum of the global plus local lateral wind moment computed according to the indicated provisions of the Guide Specification to each flange according to the relative lateral stiffness of each flange.

These constructibility checks and their associated loads are mentioned in other Discussions in this Guide, such as the Discussion of Article 3.4.2.1, the Discussion of Article 3.8.1.2.2, and the Discussion of Article 6.10.3.1. The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures and the Reference Manual for NHI Course 130102, Engineering for Structural Stability in Bridge Construction provide discussion of constructibility checks and the associated loads. The NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge provides illustrative examples of the application of these wind-load checks using the aforementioned Guide Specification provisions to the inactive and active work zone conditions for a routine steel I-girder bridge.

Sections 3.5, 6.3.2.10.2.1, 6.5.3.1, 6.5.3.6, and 6.5.6.5.1 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures also provide extensive and helpful discussions of how to approach the evaluation of wind loads during construction. These discussions focus on investigations into the possible need to provide lateral bracing to help resist wind loads and limit lateral displacements of the girders prior to the placement of the concrete deck and an example calculation is included.

The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

4.6.2.8 Seismic Lateral Load Distribution

4.6.2.8.1 Applicability

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of this Article are only applicable for bridges in Seismic Zones, 2, 3, or 4. For the purposes of this Guide, the definition of routine steel I-girder bridges only includes bridges in

Seismic Zone 1. Consequently, detailed discussion of this Article is beyond the scope of this Guide.

4.6.2.8.2 Design Criteria

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of this Article are only applicable for bridges in Seismic Zones, 2, 3, or 4. For the purposes of this Guide, the definition of routine steel I-girder bridges only includes bridges in Seismic Zone 1. Consequently, detailed discussion of this Article is beyond the scope of this Guide.

4.6.2.8.3 Load Distribution

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of this Article are only applicable for bridges in Seismic Zones, 2, 3, or 4. For the purposes of this Guide, the definition of routine steel I-girder bridges only includes bridges in Seismic Zone 1. Consequently, detailed discussion of this Article is beyond the scope of this Guide.

4.6.2.9 Analysis of Segmental Concrete Bridges

4.6.2.9.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article only apply to the analysis of segmental concrete bridges, and do not apply to the design of steel I-girder bridges.

4.6.2.9.2 Strut-and-Tie Models

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article only apply to the analysis of segmental concrete bridges, and do not apply to the design of steel I-girder bridges.

4.6.2.9.3 Effective Flange Width

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article only apply to the analysis of segmental concrete bridges, and do not apply to the design of steel I-girder bridges.

4.6.2.9.4 Transverse Analysis

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article only apply to the analysis of segmental concrete bridges, and do not apply to the design of steel I-girder bridges.

4.6.2.9.5 Longitudinal Analysis

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article only apply to the analysis of segmental concrete bridges, and do not apply to the design of steel I-girder bridges.

4.6.2.9.5a General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article only apply to the analysis of segmental concrete bridges, and do not apply to the design of steel I-girder bridges.

4.6.2.9.5b Erection Analysis

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article only apply to the analysis of segmental concrete bridges, and do not apply to the design of steel I-girder bridges.

4.6.2.9.5c Analysis of the Final Structural System

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article only apply to the analysis of segmental concrete bridges, and do not apply to the design of steel I-girder bridges.

4.6.2.10 Equivalent Strip Widths for Box Culverts

4.6.2.10.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article only apply to the analysis of concrete box culverts, and do not apply to the design of steel I-girder bridges. The provisions of this Article do not even apply to the design of decks for routine steel I-girder bridges, or the decks of any other girder bridge type.

4.6.2.10.2 Case 1: Traffic Travels Parallel to Span

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article only apply to the analysis of concrete box culverts, and do not apply to the design of steel I-girder bridges. The provisions of this Article do not even apply to the design of decks for routine steel I-girder bridges, or the decks of any other girder bridge type.

4.6.2.10.3 Case 2: Traffic Travels Perpendicular to Span

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article only apply to the analysis of concrete box culverts, and do not apply to the design of steel I-girder bridges. The provisions of this Article do not even apply to the design of decks for routine steel I-girder bridges, or the decks of any other girder bridge type.

4.6.2.10.4 Precast Box Culverts

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article only apply to the analysis of concrete box culverts, and do not apply to the design of steel I-girder bridges. The provisions of this Article do not even apply to the design of decks for routine steel I-girder bridges, or the decks of any other girder bridge type.

4.6.3 Refined Methods of Analysis

4.6.3.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

A routine steel I-girder bridge, as defined for the purposes of this Guide, is configured in such a manner that refined methods of analysis are not required for its design; instead, routine steel I-girder bridges can, and should, be designed using approximate methods of analysis, specifically line girder analysis methods. As a result, the provisions of this Article do not apply to the design of routine steel I-girder bridges.

Note that there are several commercial line girder analysis programs available to help automate and streamline the analysis and design of routine steel I-girder bridges, including NSBA's LRFD Simon line-girder analysis and design program. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

Refined methods of analysis are formally defined in the AASHTO LRFD BDS as "Methods of structural analysis that consider the entire superstructure as an integral unit and provide the required deflections and actions." More to the point, refined methods of analysis, in the context of girder bridges (or "beam-slab bridges"), directly model the girders, the cross-frames/diaphragms, the deck, and their interaction. In a refined analysis model, the distribution of vertical loads (gravity loads) is determined by consideration of the relative stiffness of the girders, cross-frames/diaphragms, and deck. Examples of refined methods of analysis for steel girder bridges include 2D grid or grillage analysis, 2D plate-and-eccentric-beam analysis, and 3D finite element analysis.

Approximate methods of analysis, on the other hand, typically consider only an individual girder, isolated from the rest of the structural system. The distributions of dead loads such as the weight of the wet concrete deck, future wearing surface, and barrier rails are typically determined using approximations such as uniform distribution of the deck weight to the girders, or semi-arbitrary percentage distribution factors. The distribution of live loads is typically determined using approximate live load distribution factors such as those found in Article 4.6.2.2 and its associated sub-Articles. The most common example of an approximate method of analysis for steel girder bridges is line girder analysis.

A routine steel I-girder bridge could be analyzed using a refined method of analysis; doing so would result in a more refined, theoretically less conservative distribution of loads and a more refined, theoretically less conservative determination of design force effects in individual structural elements such as the girders and cross-frames. However, the use of refined methods of analysis takes more time and effort on the part of the designer, and involves the use of more complicated analysis models, which are more prone to the introduction of inadvertent modeling errors, and which are more difficult to check. Approximate methods of analysis, specifically line girder analysis, are simpler to use, involving much less effort and time on the part of the designer. Line girder analysis is also less complicated, less error-prone, and easier to check. While the use of a line girder analysis may introduce some measure of conservatism, particularly in the distribution of live loads, this additional conservatism is not excessive and is considered well within the standards of acceptability in the bridge design industry. The extra refinement associated with the use of a refined method of analysis versus an approximate method of analysis is simply not worth the additional analysis time, cost, and complexity for the design of routine steel I-girder bridges; the extra time and effort that would be expended performing a refined analysis for a routine steel I-girder bridge would be far better spent on other activities, such as optimizing the framing plan or the girder sizing, preparing a clear and well-laid out set of plans, and/or performing robust checking and quality control reviews.

Finally, and importantly, using a refined method of analysis to primarily decrease conservatism in live load distribution could lead a designer to reduce girder flange or web sizes during the initial design of a bridge. This could result in difficulties later when the Owner-agency performs periodic routine load rating analyses of the bridge. For the sake of practicality, most Owner-agencies default to using line girder analysis methods for these load rating analyses – they have hundreds or thousands of bridges to load rate each year and cannot afford to perform labor-intensive refined analyses when line girder analysis methods would suffice. It is problematic when a bridge exhibits an insufficient load rating due solely to the minor conservatism of line girder analysis methods, forcing the Owner-agency to invest limited resources in performing a refined analysis to demonstrate that a bridge has sufficient load-carrying capacity.

4.6.3.2 Decks

4.6.3.2.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

A routine steel I-girder bridge, as defined for the purposes of this Guide, is configured in such a manner that refined methods of analysis are not required for its design; instead, routine steel I-girder bridges can, and should, be designed using approximate methods of analysis, specifically line girder analysis methods. As a result, the provisions of this Article do not apply to the design of routine steel I-girder bridges. See the Discussion of Article 4.6.3.1 for more information about refined versus approximate methods of analysis.

Note that there are several commercial line girder analysis programs available to help automate and streamline the analysis and design of routine steel I-girder bridges, including NSBA's LRFD Simon line-girder analysis and design program. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

4.6.3.2.2 Isotropic Plate Model

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

A routine steel I-girder bridge, as defined for the purposes of this Guide, is configured in such a manner that refined methods of analysis are not required for its design; instead, routine steel I-girder bridges can, and should, be designed using approximate methods of analysis, specifically line girder analysis methods. As a result, the provisions of this Article do not apply to the design of routine steel I-girder bridges. See the Discussion of Article 4.6.3.1 for more information about refined versus approximate methods of analysis.

Note that there are several commercial line girder analysis programs available to help automate and streamline the analysis and design of routine steel I-girder bridges, including NSBA's LRFD Simon line-girder analysis and design program. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

4.6.3.2.3 Orthotropic Plate Model

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

A routine steel I-girder bridge, as defined for the purposes of this Guide, is configured in such a manner that refined methods of analysis are not required for its design; instead, routine steel I-girder bridges can, and should, be designed using approximate methods of analysis, specifically line girder analysis methods. As a result, the provisions of this Article do not apply to the design of routine steel I-girder bridges. See the Discussion of Article 4.6.3.1 for more information about refined versus approximate methods of analysis.

Note that there are several commercial line girder analysis programs available to help automate and streamline the analysis and design of routine steel I-girder bridges, including NSBA's LRFD Simon line-girder analysis and design program. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

4.6.3.2.4 Refined Orthotropic Deck Model

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

A routine steel I-girder bridge, as defined for the purposes of this Guide, is configured in such a manner that refined methods of analysis are not required for its design; instead, routine steel I-girder bridges can, and should, be designed using approximate methods of analysis, specifically line girder analysis methods. As a result, the provisions of this Article do not apply to the design of routine steel I-girder bridges. See the Discussion of Article 4.6.3.1 for more information about refined versus approximate methods of analysis.

Note that there are several commercial line girder analysis programs available to help automate and streamline the analysis and design of routine steel I-girder bridges, including NSBA's LRFD Simon line-girder analysis and design program. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

4.6.3.3 Beam-Slab Bridges

4.6.3.3.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

A routine steel I-girder bridge, as defined for the purposes of this Guide, is configured in such a manner that refined methods of analysis are not required for its design; instead, routine steel I-girder bridges can, and should, be designed using approximate methods of analysis, specifically line girder analysis methods. As a result, the provisions of this Article do not apply to the design of routine steel I-girder bridges. See the Discussion of Article 4.6.3.1 for more information about refined versus approximate methods of analysis.

Note that there are several commercial line girder analysis programs available to help automate and streamline the analysis and design of routine steel I-girder bridges, including NSBA's LRFD Simon line-girder analysis and design program. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

4.6.3.3.2 2D Grid and Plate and Eccentric Beam Analyses of Curved and/or Skewed Steel I-Girder Bridges

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

A routine steel I-girder bridge, as defined for the purposes of this Guide (a straight bridge with little or no skew), is configured in such a manner that refined methods of analysis are not required for its design; instead, routine steel I-girder bridges can, and should, be designed using approximate methods of analysis, specifically line girder analysis methods. As a result, the provisions of this Article (which discuss 2D grid and plate and eccentric beam analyses of curved and/or skewed steel I-girder bridges) do not apply to the design of routine steel I-girder bridges. See the Discussion of Article 4.6.3.1 for more information about refined versus approximate methods of analysis.

Note that there are several commercial line girder analysis programs available to help automate and streamline the analysis and design of routine steel I-girder bridges, including NSBA's LRFD Simon line-girder analysis and design program. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

4.6.3.3.3 Curved Steel Bridges

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

A routine steel I-girder bridge, as defined for the purposes of this Guide (a straight bridge with little or no skew), is configured in such a manner that refined methods of analysis are not required for its design; instead, routine steel I-girder bridges can, and should, be designed using approximate methods of analysis, specifically line girder analysis methods. As a result, the provisions of this Article (which discuss 2D grid and plate and eccentric beam analyses of curved and/or skewed steel I-girder bridges) do not apply to the design of routine steel I-girder bridges. See the Discussion of Article 4.6.3.1 for more information about refined versus approximate methods of analysis.

Note that there are several commercial line girder analysis programs available to help automate and streamline the analysis and design of routine steel I-girder bridges, including NSBA's LRFD Simon line-girder analysis and design program. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

4.6.3.3.4 Cross-Frames and Diaphragms

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

A routine steel I-girder bridge, as defined for the purposes of this Guide (a straight bridge with little or no skew), is configured in such a manner that refined methods of analysis are not required for its design; instead, routine steel I-girder bridges can, and should, be designed using approximate methods of analysis, specifically line girder analysis methods. As a result, the provisions of this Article do not apply to the design of routine steel I-girder bridges. See the Discussion of Article 4.6.3.1 for more information about refined versus approximate methods of analysis.

Note that there are several commercial line girder analysis programs available to help automate and streamline the analysis and design of routine steel I-girder bridges, including NSBA's LRFD Simon line-girder analysis and design program. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

4.6.3.3.4a 2D Grid and Plate and Eccentric Beam Analyses

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

A routine steel I-girder bridge, as defined for the purposes of this Guide (a straight bridge with little or no skew), is configured in such a manner that refined methods of analysis are not required for its design; instead, routine steel I-girder bridges can, and should, be designed using approximate methods of analysis, specifically line girder analysis methods. As a result, the provisions of this Article (which relate to 2D grid and plate and eccentric beam analyses) do not apply to the design of routine steel I-girder bridges. See the Discussion of Article 4.6.3.1 for more information about refined versus approximate methods of analysis.

Note that there are several commercial line girder analysis programs available to help automate and streamline the analysis and design of routine steel I-girder bridges, including NSBA's LRFD Simon line-girder analysis and design program. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

4.6.3.3.4b 3D Analyses

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

A routine steel I-girder bridge, as defined for the purposes of this Guide (a straight bridge with little or no skew), is configured in such a manner that refined methods of analysis are not required for its design; instead, routine steel I-girder bridges can, and should, be designed using approximate methods of analysis, specifically line girder analysis methods. As a result, the provisions of this Article (which relate to 3D analyses) do not apply to the design of routine steel I-girder bridges. See the Discussion of Article 4.6.3.1 for more information about refined versus approximate methods of analysis.

Note that there are several commercial line girder analysis programs available to help automate and streamline the analysis and design of routine steel I-girder bridges, including NSBA's LRFD

Simon line-girder analysis and design program. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

4.6.3.3.4c Equivalent Axial Rigidity of Single-Angle and Tee-Section Cross-Frame Members

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

A routine steel I-girder bridge, as defined for the purposes of this Guide (a straight bridge with little or no skew), is configured in such a manner that refined methods of analysis are not required for its design; instead, routine steel I-girder bridges can, and should, be designed using approximate methods of analysis, specifically line girder analysis methods. As a result, the provisions of this Article (which relate to 2D grid and plate and eccentric beam analyses and 3D analyses) do not apply to the design of routine steel I-girder bridges. See the Discussion of Article 4.6.3.1 for more information about refined versus approximate methods of analysis.

Note that there are several commercial line girder analysis programs available to help automate and streamline the analysis and design of routine steel I-girder bridges, including NSBA's LRFD Simon line-girder analysis and design program. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

4.6.3.4 Cellular and Box Bridges

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article does not apply to the design of routine steel I-girder bridges, or to any beam-slab, girder-type bridge.

4.6.3.5 Truss Bridges

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article does not apply to the design of routine steel I-girder bridges, or to any beam-slab, girder-type bridge.

4.6.3.6 Arch Bridges

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article does not apply to the design of routine steel I-girder bridges, or to any beam-slab, girder-type bridge.

4.6.3.7 Cable-Stayed Bridges

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article does not apply to the design of routine steel I-girder bridges, or to any beam-slab, girder-type bridge.

4.6.3.8 Suspension Bridges

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article does not apply to the design of routine steel I-girder bridges, or to any beam-slab, girder-type bridge.

4.6.4 Redistribution of Negative Moments in Continuous Beam Bridges

4.6.4.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Allowing redistribution of negative moments in multi-span continuous steel I-girder bridges can potentially produce more economical designs, but the associated analysis and design considerations are unfamiliar to most designers and most Owner-agencies currently do not permit or encourage the use of moment redistribution methods. As a result, the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges.

A basic explanation of moment distribution methods and their associated advantages and disadvantages is provided below for information only.

In conventional elastic analysis and design, moment and shear envelopes are typically determined by elastic analysis with no consideration of redistribution due to the effects of yielding. Even if localized yielding at some section is permitted for a specific girder design check, redistribution of moments and shears to account for such local yielding is not addressed by the basic provisions of Section 6 of the AASHTO LRFD BDS. Under these provisions, when performing conventional elastic analysis and design, the girder sections must be proportioned to provide resistance equal to or greater than that required by the moment and shear envelopes determined by the elastic analysis. The requirement to meet these moment and shear demands can lead to designs which are less than optimal from a performance or economic standpoint. For instance, the designer may choose to use oversized sections or reduce beam spacing and add more beams to the cross section. In welded beams, multiple flange transitions might be added, resulting in increased fabrication costs.

On the other hand, accounting for the redistribution of moments (where appropriate) can make it possible to eliminate such details by using prismatic sections along the entire length of the bridge or between field splices, thus providing fabrication economies and improving the overall fatigue resistance. This is made possible by removing restrictions on the flexural resistance in the regions adjacent to interior piers from which moments are redistributed at both the service and strength limit states by accounting for the strength and ductility of the pier sections directly within the procedures used to calculate the redistribution moments.

For straight continuous-span steel I-girder bridges, the optional Appendix B6 of the AASHTO LRFD BDS provides both an approximate procedure and a refined method to calculate the redistribution moments (see the Discussion of Appendix B6 in this Guide). The provisions of Appendix B6 may be applied only to straight continuous-span I-section members whose support lines are not skewed more than 10 degrees from radial and along which there are no staggered cross-frames. Cross-sections throughout the unbraced lengths immediately adjacent to interior-pier sections from which moments are redistributed must also satisfy certain specified restrictions to provide adequate robustness to redistribute the moments.

Although members in routine steel I-girder bridges covered by this Guide may satisfy these restrictions, the use of moment redistribution should only be undertaken with the full knowledge and consent of the Owner, and only with a full understanding of the implications. While use of this method can potentially result in a more economical design in terms of smaller/lighter girder/beam sections, the necessary analysis is somewhat more time-consuming than that for a design in which the section is designed to remain elastic since commercial software packages do not currently include the capability for automating the associated calculations.

For further information on the provisions of Appendix B6, consult Section 6.5.6.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

4.6.4.2 Refined Method

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges. See the Discussion of Article 4.6.4.1 for more information.

4.6.4.3 Approximate Procedure

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges. See the Discussion of Article 4.6.4.1 for more information.

4.6.5 Stability

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The investigation of stability utilizing large deflection theory is not applicable to routine steel I-girder bridge superstructures but may be applicable for the design of substructures (specifically tall piers).

The design of most steel girder bridges, including routine steel I-girder bridges, is typically based on small-deflection theory. Small-deflection theory is the basis for methods of analysis where the effects of deformation upon force effects in the structure are neglected. In small-deflection theory analyses, second-order geometric nonlinear behavior is not considered. Instead, it is assumed that the deformations of the structure are small enough that they do not lead to second-order amplification of member loads. This is a perfectly rational and reasonable assumption for the design of routine steel I-girder bridges.

There are provisions in the specifications, specifically in Article 6.10.1.6, where second-order compression-flange lateral bending stresses are approximated using a simple formula to amplify the first-order values. However, these provisions are specifically intended only for amplification of compression-flange lateral bending stresses due to torsion, such as those that occur due to the effect of deck overhang loads acting on exterior (fascia) girders (see the Discussions of Article 6.10.1.6 and the associated sub-Articles of Article 6.10.3 in this Guide).

Cases where it may be appropriate to base the analysis of a steel girder bridge on large deflection theory include structure types which have been specifically excluded from the definition of routine steel I-girder bridges. Examples include, but are not limited to, narrow, slender steel I-girder superstructures with three or fewer girders during the deck placement (e.g., in a phased construction situation), which may experience global amplification of lateral-torsional deformations and potentially be subject to global lateral-torsional buckling. Again, the definition of a routine steel I-girder bridge for the purposes of this Guide includes limitations which preclude concern about this type of behavior (in this case, the specific requirement that the cross-section include four or more girders with no consideration of phased construction).

4.6.6 Analysis for Temperature Gradient

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Consideration of temperature gradient effects is unnecessary for the design of routine steel I-girder bridges.

Typically, the main reason for performing a temperature gradient analysis in a steel girder bridge is to evaluate the potential for uplift at the bearings. This is generally more of a concern in narrow curved and/or skewed steel girder bridges. It is highly unlikely that a temperature gradient loading would produce uplift in a bridge meeting the definition of a routine steel I-girder bridge.

It is also highly unlikely that temperature gradient loading would contribute to a controlling load case in terms of stresses or forces in a routine steel I-girder bridge. Consider the recommended load factors for temperature gradient presented in the AASHTO LRFD BDS: 0.0 for the strength and extreme event limit states, 1.0 at the service limit state when live load is not present, and 0.50 at the service limit state when live load is considered. It is difficult to imagine a situation where the effects of temperature gradient on member forces or stresses would contribute to a controlling load combination when such load factors are used.

Most guideline documents either explicitly or implicitly recommend neglecting analysis of the temperature gradient effect for the design of bridges which meet the definition of a routine steel I-girder bridge. For example, the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, explicitly neglects consideration of the effects of temperature gradient loading. The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures discusses the temperature gradient (TG) load case in general terms in Chapter 3, Loads and Load Factors, mentions it several times in Chapter 5, Concrete Girder Superstructures, but does not mention this load case in Chapter 6, Steel Girder Superstructures. The Reference Manual for NHI Course 130095, Analysis and Design of Skewed and Curved Steel Bridges with LRFD, mentions temperature gradient specifically as a loading which can potentially contribute to uplift in narrow curved and/or skewed steel girder bridges, and which should be considered in determining bearing reactions. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

Finally, it should also be noted that the effects of temperature gradient cannot be captured in a typical line girder analysis, and that line girder analysis is widely acknowledged as being perfectly appropriate for the design of routine steel I-girder bridges.

4.7 DYNAMIC ANALYSIS

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of Article 4.7 and its associated sub-Articles are intended to address a wide variety of dynamic analyses which may be applicable to the design of bridges. For the purposes of this Guide, it is helpful to group these potential applications for discussion.

Dynamic analysis may be appropriate for evaluation of vehicle- and/or wind-induced vibrations of certain types of bridge superstructures. However, this type of analysis is typically only necessary for extremely flexible bridges. Bridges which meet the description of a routine steel I-girder bridge, as defined for the purposes of this Guide, generally exhibit sufficient stiffness to avoid harmful dynamic response to vehicle or wind loading. Consequently, dynamic analysis of routine steel I-girder bridges is not necessary to investigate vehicle- or wind-induced vibrations.

Dynamic analysis is also appropriate for the evaluation of the response of bridges to various lateral loading cases, such as wind loading, vessel collision, blast forces, or seismic loading. Vessel collision and blast loading are considered beyond the scope of this Guide; designers faced with the need to evaluate these types of loading conditions are encouraged to consult with experienced senior bridge engineers to define the scope and approach for such analyses. Wind loading typically does not induce harmful dynamic response in superstructures proportioned and configured in a manner that would meet the description of a routine steel I-girder bridge as defined by this Guide; typically, wind loading is more of a concern for particularly tall or long span bridges. Seismic loading can be a concern for virtually any bridge, depending on the nature and magnitude of that loading. However, the routine steel I-girder bridges covered by this Guide are assumed, by definition, to be in Seismic Zone 1. Dynamic analysis for seismic loads is only applicable for bridges in Seismic Zones, 2, 3, or 4; therefore, detailed discussion of dynamic analysis for seismic loading is beyond the scope of this Guide. Overall, dynamic analysis of routine steel I-girder bridges is not necessary to evaluate the effects of lateral loading.

4.7.4 Analysis for Earthquake Loads

4.7.4.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article specifies minimum analysis requirement for seismic effects, based on the seismic zone in which the bridge resides. Only bridges in Seismic Zone 1 meet the definition of a routine steel I-girder bridge for the purposes of this Guide, so only the provisions related to bridges in Seismic Zone 1 apply. Note that this Article states that bridges in Seismic Zone 1 are subject to the provisions of Articles 4.7.4.4 and 3.10.9.

4.7.4.2 Single-Span Bridges

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article states that seismic analysis is not required for single-span bridges, regardless of seismic zone. The Article also references Article 3.10.9 for minimum forces requirements for the design of connections between the bridge superstructure and abutments. While these requirements apply to the routine steel I-girder bridges covered by this Guide, the design of bearings and substructures is considered beyond the scope of superstructure design.

4.7.4.3 Multispan Bridges

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article provides the minimum seismic analysis requirements for multi-span bridges. Only bridges in Seismic Zone 1 meet the definition of a routine steel I-girder bridge for the purposes of this Guide, and the provisions of this Article indicate that no seismic analysis is required for bridges in Seismic Zone 1. As a result, the associated sub-Articles under Article 4.7.3 are not discussed in this Guide since none of them are applicable.

Note that Article 4.7.4.1 states that bridges in Seismic Zone 1 are subject to the provisions of Articles 4.7.4.4 and 3.10.9 (see the Discussion of Article 4.7.4.1 in this Guide).

4.7.4.4 Minimum Support Length Requirements

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article specifies the minimum support length requirements for bridges. Only bridges in Seismic Zone 1 meet the definition of a routine steel I-girder bridge for the purposes of this Guide. For bridges in Seismic Zone 1, this Article states that the requirements of Article 4.7.4.3 do not apply, and only the provisions of this Article (4.7.4.4) need to be considered.

This Article provides a simple formula for the nominal empirical minimum support length, N, measured normal to the centerline of bearing at expansion bearings without restrainers, STUs, or dampers. The Article also includes Table 4.7.4.4-1, which specifies the percentage of the minimum support length, N, which must actually be provided, as a function of the Acceleration Coefficient, A_s (see the Discussion of Article 3.10.4.2 in this Guide); otherwise, longitudinal restrainers must be provided in accordance with Article 3.10.9.5.

4.7.4.5 *P*-Δ Requirements

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

This Article discusses the design of columns and piers to limit seismic displacements so that P-Δ effects will not significantly affect the response of the bridge during an earthquake, which is not applicable to bridges in Seismic Zone 1 and is also beyond the scope of superstructure design.

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6.1 SCOPE

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article provides the overall scope of what types of steel components and systems are addressed by the design provisions given in Section 6. The Article also discusses the limitations of the design provisions as they relate specifically to horizontally curved-girder systems, which are not applicable to the routine steel-girder bridges covered by this Guide. The Article also points to Appendix C6, which provides a helpful outline of the basic steps for the design of steel-girder bridges; note that only the steps shown in the outline for I-girder bridges are applicable to the routine steel-girder bridges covered by this Guide (see the Discussions of Articles C6.1 through C6.3 in this Guide).

The Commentary for this Article provides a discussion of the overall organization of Section 6. Articles 6.10 and 6.13 are the Articles primarily applicable to routine steel I-girder bridges; portions of Article 6.8, 6.9, and 6.12 may also be partially or conditionally applicable, or beyond the scope of superstructure design, as discussed further below. The application of advanced analysis methods, as discussed in the Commentary, is typically neither necessary nor recommended for the routine steel I-girder bridges covered by this Guide.

6.2 DEFINITIONS

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article provides an alphabetic listing of the specific definitions for commonly used terms throughout Section 6. Designers should refer to these definitions whenever a term is encountered in Section 6 to clearly understand the specific meaning that the code writers intended to apply to that term. Note that many, but not all, of the definitions will apply to routine steel I-girder bridges.

6.3 NOTATION

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article provides an alphabetical listing of the definitions for the variables used in the text and in the equations throughout Section 6. Some of the variables are re-used and have multiple definitions. The Article number where a specific definition for a given variable first appears in Section 6 is provided in parentheses at the end of the definition(s). Note that many, but not all, of the variables and their definitions will apply to the design of routine steel I-girder bridges.

6.4 MATERIALS

6.4.1 Structural Steels

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article provides the material specification names and minimum material properties for the applicable plate and rolled steel used for steel bridges. The material specification for structural steel for bridges is ASTM A709/A709M, with the "M" referring to the metric version of the specification, which is not applicable. The AASHTO M270M/M270 material specification is equivalent to the ASTM A709/A709M specification but is typically several versions behind the ASTM specification. The specification that is applicable depends on the specific Owner-agency standards.

Only Grades 36, 50, 50S, and 50W are considered applicable for the routine steel I-girder bridges covered by this Guide, with Grades 50 and 50W being the standard grades of structural plate steel specified for the girders in routine plate-girder bridges (with the "W" referring to weathering steel), and Grades 50S and 50W being the only grades of structural steel that can be specified for the rolled wide-flange beams in routine rolled-beam bridges (with the "S" referring to shape). There is virtually no price advantage associated with the use of Grade 36 steels, so it is rarely if ever used for the design of plate girders or rolled beams in modern designs. "HPS" designates "high performance steel" grades which have enhanced weldability and toughness; however, the use of HPS 50W is typically not warranted since there have been few weldability problems reported in the non-HPS Grade 50 and 50W steels, and the enhanced toughness is generally of more value for certain nonredundant steel tension members with low redundancy such as tension ties in tied-arch bridges. Other structural steels listed in this Article are not applicable to routine steel-girder bridges.

Note that rolled wide-flange beams used as diaphragms and structural tees used as cross-frame members in routine steel-girder bridges should be specified as Grade 50S or 50W; structural tees are typically split by the fabricator from rolled wide-flange beams. Angles and channels used as cross-frame or diaphragm members are available as either Grade 36, 50, or 50W and should be specified accordingly. Detail steel, e.g., steel used to fabricate connection plates and stiffeners, is typically specified as Grade 50 or 50W but Grade 36 is sometimes used; in some cases, Owner-agency policy may specify that Grade 36 is permitted to be substituted in place of Grade 50 in detail steel applications.

Table 6.4.1-1 provides the specified minimum yield strength, F_y, and the specified minimum tensile strength, F_u, for Grades 36, 50, 50S, and 50W, and the ranges of plate thicknesses available in Grades 36, 50, and 50W.

For additional information, consult the NSBA's Steel Bridge Design Handbook – Chapter 1: Bridge Steels and Their Mechanical Properties.

6.4.2 Pins, Rollers, and Rockers

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Pins, rollers, and rockers are found on older existing bridges, and are not typically used on modern steel bridges, including the routine steel I-girder bridges covered by this Guide. Therefore, the provisions in this Article are considered not applicable to such bridges.

6.4.3 Bolts, Nuts, and Washers

6.4.3.1 High-Strength Structural Fasteners

6.4.3.1.1 High-Strength Bolts

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article defines the material specification for high-strength bolts used as structural fasteners, which are used in bolted field splices and in cross-frame or diaphragm connections in routine steel I-girder bridges. High-strength bolts are to conform to either the ASTM F3125/3125M specification, with the “M” referring to the metric version of the specification which is not applicable, or the ASTM F3148 Specification.

Grade A325 bolts are the most common high-strength bolts under the ASTM F3125 specification but Grade A490 bolts may sometimes be used to reduce the number of bolts required in the connection if permitted by Owner-agency policy. The twist-off equivalents of Grade A325 and A490 bolts under the ASTM F3125 specification, i.e., Grade F1852 and F2280 respectively, may alternatively be used depending on the preferences of the Owner-agency.

The ASTM F3148 standard covers high-strength bolt assemblies with fixed-spline drives that are intended to be installed with a torque-and-angle installation method, also referred to as the combined installation method. ASTM F3148 bolts look like a twist-off bolt, however, the splined end is not intended to shear off. The presence of the splined end is for torque reaction for one-side tensioning, which is a distinct advantage.

ASTM F3125 Grade A490 and F2280 bolts only have enough ductility to undergo one pretensioning operation (i.e., they cannot be installed and fully tightened, loosened, and then retightened or reused). Some owners discourage or prohibit the use of ASTM F3125 Grade A490 and F2280 bolts for this reason.

The Engineer should refer to the latest versions of ASTM F3125/3125M and ASTM F3148 for permissible corrosion-resistant coating options. These options, as of 2022, are summarized in the Commentary for this Article. Type 3 bolts are to be used with weathering steels. Note that galvanizing is not an acceptable option for high-strength bolts with a specified minimum tensile strength of 150 ksi (i.e., ASTM F3125 Grade A490 and F2280 bolts). The rotational capacity testing of the fastener assemblies mentioned in the Commentary is performed by the bolt manufacturer.

Table 6.4.3.1.1-1 provides the specified minimum tensile strength of the bolts, F_{ub}, which is used to compute the factored shear resistance of the bolts according to the provisions of Article 6.13.2.7

(see the Discussion of Article 6.13.2.7 in this Guide). Note that the specified minimum tensile strength of ASTM F3148 bolts of 144 ksi is in-between the strength levels of ASTM F3125 Grade A325 and A490 bolts.

For further information on high-strength bolts, including installation provisions and verification procedures, consult the AISC Design Guide 17 High Strength Bolts - A Primer for Engineers, the RCSC Specifications for Structural Joints Using High-Strength Bolts available from the Research Council on Structural Connections (RCSC), the AASHTO *LRFD Steel Bridge Fabrication Specifications* and the applicable Owner-agency standards.

6.4.3.1.2 Nuts Used with High-Strength Bolts

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article simply indicates that the nuts recommended or suitable for use with the various grades of high-strength bolts described above are listed in the ASTM F3125 or ASTM F3148 specification, as applicable.

6.4.3.1.3 Washers Used with High-Strength Bolts

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article simply indicates that the hardened washers recommended or suitable for use with the various grades of high-strength bolts described above are listed in the ASTM F3125 or ASTM F3148 specification, as applicable. The need for washers and the selection of them is typically done by the fabricator.

6.4.3.1.4 Direct Tension Indicators

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article provides the requirements for direct tension indicators, or so-called "DTIs" which may be used, depending on the preferences and policies of the Owner-agency, in conjunction with high-strength bolts, nuts, and washers to verify the required bolt installation tension. DTIs are washers which include mechanical features (typically small arch-shaped protrusions) which compress in response to the pretension developed in the bolt. When correctly calibrated, the amount of pretension can be determined by measuring the gap remaining between the washer and the connected element.

The material specification for DTIs is ASTM A959/A959M, with the "M" referring to the metric version of the specification which is not applicable. Two alternative DTIs known as captive DTI/nuts and self-indicating DTIs may be used if permitted by Owner-agency policy. When used, refer to the reference documents cited in the Discussion of Article 6.4.3.1.1 for further information on DTIs and their specific installation provisions and verification procedures.

6.4.3.2 Low-Strength Steel Bolts

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article provides the requirements for low-strength steel bolts, which may be used to connect non-structural items in routine steel-girder bridges, if permitted by Owner-agency policy. The material specification for these bolts is the ASTM A307 specification, which covers two grades of bolts (Grades A and B). The specified minimum tensile strength of the bolts, F_{ub}, is given as 60 ksi, which is used to compute the factored shear resistance of the bolts according to the provisions of Article 6.13.2.7 using the equation for threads included in the shear plane (see the Discussion of Article 6.13.2.7 in this Guide).

6.4.3.3 Fasteners for Structural Anchorage

6.4.3.3.1 Anchor Rods

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article defines the material specification for anchor rods, i.e., ASTM F1554, which are typically only used at bearings in most routine steel-girder bridges. Anchor rods are also commonly referred to as “anchor bolts”; the two terms are considered synonymous.

6.4.3.3.2 Nuts Used with Anchor Rods

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article simply indicates the nuts that must be used with ASTM F1554 anchor rods (also sometimes called “anchor bolts”), which are typically only used at bearings in most routine steel-girder bridges.

6.4.4 Stud Shear Connectors

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article provides the specified minimum yield strength, F_y, and specified minimum tensile strength, F_u, of stud shear connectors, which are used in routine steel-girder bridges to develop composite action by preventing slip between the concrete deck and the steel beam or girder. F_u is used in the computation of the nominal shear resistance, Q_n, of a stud shear connector at the strength limit state in Eq. 6.10.10.4.3-1 (see the Discussion of Article 6.10.10.4.3 in this Guide).

Consult the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* for further information on the specifications for material, manufacturing, physical properties, certification, and welding of stud shear connectors.

6.4.5 Weld Metal

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article refers to the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* (BWC) for the specific requirements for weld metal, including the specification of the weld metal and flux by the appropriate AWS designation. Virtually all routine steel I-girder bridges utilize structural steel welds. Using AWS classifications, the BWC prescribes which consumables may be used with various base metals and the rules for their use. Fabricators choose welding processes and associated consumables to be used on the bridge and develop and follow welding procedure specifications (WPSs) that conform to BWC requirements.

Consult the FHWA Bridge Welding Reference Manual for additional information on weld metal, welding processes, and the appropriate designations of welding consumables.

6.4.6 Cast Metal

6.4.6.1 Cast Steel and Ductile Iron

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides the appropriate material specifications for cast steel and ductile iron castings. Routine steel I-girder bridges do not typically contain castings and so this Article is designated as not applicable.

6.4.6.2 Malleable Castings

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides the appropriate material specification and the minimum specified yield strength for malleable castings. Routine steel I-girder bridges do not typically contain castings and so this Article is designated as not applicable.

6.4.6.3 Cast Iron

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides the appropriate material specification for cast iron castings. Routine steel I-girder bridges do not typically contain castings and so this Article is designated as not applicable.

6.4.7 Stainless Steel

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article contains requirements related to the material specifications for stainless steel. Routine steel I-girder bridges do not typically utilize stainless steel for structural members such as girders, cross-frames, or diaphragms, but the provisions may apply for bearings with stainless steel sliding surfaces.

6.4.8 Cables

6.4.8.1 Bright Wire

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides the appropriate material specification for bright wire used in bridge cables. Routine steel I-girder bridges do not make use of cables and so this Article is designated as not applicable.

6.4.8.2 Galvanized Wire

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides the appropriate material specification for galvanized wire used in bridge cables. Routine steel I-girder bridges do not make use of cables and so this Article is designated as not applicable.

6.4.8.3 Epoxy-Coated Wire

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides the appropriate material specification for epoxy-coated wire used in bridge cables. Routine steel I-girder bridges do not make use of cables and so this Article is designated as not applicable.

6.4.8.4 Bridge Strand

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides the appropriate material specification for strand used in bridge cables. Routine steel I-girder bridges do not make use of cables and so this Article is designated as not applicable.

6.4.9 Dissimilar Metals

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article provides requirements to help prevent galvanic corrosion when steel components are coupled with aluminum components in the presence of an electrolyte. Aluminum is not used in the fabrication of structural members such as girders or cross-frames/diaphragms for routine steel I-girder bridges. This Article would only be applicable to routine steel I-girder bridges in situations where aluminum components (such as sign support brackets, for example) might be fastened to the structural steel.

6.5 LIMIT STATES

6.5.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.Partially applicable.

Discussion:

This Article presents the four limit states for which structural steel-bridge components must be proportioned to satisfy the applicable design requirements specified at each of these limit states. Three of these limit states are applicable or partially applicable to routine steel I-girder bridges, as explained further in the Discussions for Articles 6.5.2, 6.5.3, 6.5.4, and 6.5.5 in this Guide.

For further information on each limit state, consult the NSBA's Steel Bridge Design Handbook – Chapter 10: Limit States, and Section 1.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures). The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.5.2 Service Limit State

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The service limit state is taken as restrictions on stress, deformation, and deck crack width under regular service conditions. This Article requires that flexural members in routine steel I-girder bridges satisfy the service limit state checks specified in Article 6.10.4 (see the Discussion of Article 6.10.4 in this Guide) to prevent objectionable permanent deformations under expected severe traffic loadings that may impair rideability (i.e., under the Service II load combination specified in Table 3.4.1-1, which applies a load factor of 1.30 to the live load force effects).

In addition, Article 6.10.1.7 (see the Discussion of Article 6.10.1.7 in this Guide) presents provisions intended to control the width of cracks in the concrete decks in the negative moment regions of multi-span continuous steel-girder bridges under regular service conditions (i.e., under the Service II load combination specified in Table 3.4.1-1). This Article also points to optional span-to-depth ratios and live-load deflection requirements specified in Article 2.5.2.6. Most Owners choose to enforce a live-load deflection requirement at the Service I limit state (which applies a load factor of 1.0 to the live load force effects); consult the applicable Owner-agency policy. See the Discussion of Article 2.5.2.6.2 in this Guide for further information on the computation of the live-load deflection.

For further background and explanation of the service limit state, consult the NSBA's Steel Bridge Design Handbook – Chapter 10: Limit States, and Section 1.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

Note that the resistance factors, ϕ, used in service limit state calculations are implicitly taken equal to 1.0 for all members and components.

6.5.3 Fatigue and Fracture Limit State

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

The fatigue limit state is taken as restrictions on stress ranges resulting from the passage of a single design truck specified in Article 3.6.1.4 occurring at the number of expected stress range cycles. The fracture limit state is taken as a set of material toughness requirements. This Article requires that components and details in routine steel I-girder bridges satisfy the fatigue limit state checks and fracture toughness considerations specified in Article 6.6 (see the Discussion of Article 6.6 in this Guide). In addition, flexural members in routine steel I-girder bridges must satisfy the additional fatigue limit state checks specified in Article 6.10.

The provisions of Article 6.13.2.10.3 for bolts subject to tensile fatigue (see the Discussion of Article 6.13.2.10.3 in this Guide) are not applicable for the routine steel I-girder bridges covered by this Guide because the typical bolted connections in such bridges are not subject to axial tension.

For further background and explanation of the fatigue and fracture limit state, consult the NSBA's Steel Bridge Design Handbook – Chapter 10: Limit States, and Section 1.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

Note that the resistance factors, ϕ, used in fatigue limit state calculations are implicitly taken equal to 1.0 for all members and components.

6.5.4 Strength Limit State

6.5.4.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The strength limit state is investigated to check that strength and both local and global stability are provided to resist the statistically significant load combinations that a bridge is expected to experience over its design life. This Article requires that steel I-girder bridges satisfy the applicable strength and stability checks in Section 6 in the final condition for the factored force effects at the strength limit state calculated using the appropriate strength load combinations specified in Table 3.4.1-1, and also for the force effects acting on the fully erected steelwork during the deck placement calculated using the special load combination specified in Article 3.4.2.1.

As can be seen by reviewing these load combinations, many of the load factors are greater than 1.0; the load factors in Table 3.4.1-1 were calculated by means of statistical analyses to envelope possible overload conditions which are considered "statistically significant" (i.e., overload conditions which have a certain probability of occurring over the anticipated life of the structure). The load factor of 1.4 in the special load combination specified in Article 3.4.2.1 for evaluating the constructibility of primary steel superstructure components for the force effects applied to the fully erected steelwork was not statistically calibrated; instead, it was selected to provide a level of strength and stability during critical construction stages (where unintended events could potentially lead to significantly larger force effects than those predicted during the design) which at least approaches that attained in the past using previous design methodologies. A similar load combination is specified in the AISC *LRFD Specification for Structural Steel Buildings*.

For further background and explanation of the strength limit state, consult the NSBA's Steel Bridge Design Handbook – Chapter 10: Limit States, and Section 1.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.5.4.2 Resistance Factors

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article lists the resistance factors, ϕ, applied to the nominal resistance, R_n, of members and components at the strength limit state to compute the factored resistance, R_r. Some, but not all, resistance factors are applicable to routine steel I-girder bridges.

The resistance factors are implicitly taken equal to 1.0 for members and components at the service and fatigue and fracture limit states.

Designers should thoroughly review the entire list of resistance factors to identify the correct resistance factor for each specific design calculation. Designers should review the AASHTO LRFD BDS Articles associated with the use of a given resistance factor, as well as reviewing the Determination of Applicability and Discussion in this Guide that correspond to those Articles.

6.5.5 Extreme Event Limit State

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The extreme event limit state is investigated to check that the bridge can survive a specified major earthquake, flood, vessel collision, vehicle collision, or ice collision event, possibly under scoured conditions. Routine steel I-girder bridges as defined for the purposes of this Guide are assumed to be located in Seismic Zone 1 and not subject to stream flow loading, ice loading, or vessel collision loading of the superstructure. Also, the provisions of this Article are not applicable to the superstructure design for routine steel I-girder bridges located in Seismic Zone 1. Therefore, this Article is not applicable to the design of the routine steel I-girder bridges covered by this Guide.

6.6 FATIGUE AND FRACTURE CONSIDERATIONS

6.6.1 Fatigue

6.6.1.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article categorizes fatigue into load-induced fatigue (Article 6.6.1.2) and distortion-induced fatigue (Article 6.6.1.3).

For further information on fatigue design, consult the NSBA's Steel Bridge Design Handbook – Chapter 12: Design for Fatigue, the Reference Manual for NHI Course 130122, Design and Evaluation of Steel Bridges for Fatigue and Fracture, and Section 6.5.5 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples illustrating load-induced fatigue design computations, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.6.1.2 Load-Induced Fatigue

6.6.1.2.1 Application

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Applicable.

Discussion:

Load-induced fatigue is defined as fatigue effects due to the in-plane stresses for which components and details are explicitly designed. This Article indicates that the force effect to be considered in the load-induced fatigue design of components and details in routine steel I-girder bridges is the live load stress range, or the algebraic difference between the maximum and minimum fatigue live-load stresses in the component or at the detail under consideration due to the fatigue live load placed in a single lane.

In routine steel multi-span continuous I-girder bridges with shear connectors provided throughout their length and longitudinal deck reinforcement satisfying the provisions of Article 6.10.1.7 (see the Discussion of Article 6.10.1.7 in this Guide), the concrete deck may be considered effective in tension for computing the stress due to the negative (minimum) fatigue live-load moment (using the short-term modular ratio, *n*, to transform the concrete deck) when calculating the stress range. This is strongly recommended when computing the stress range at details on beams or girders in routine steel multi-span continuous I-girder bridges satisfying the preceding criteria; recognition of this behavior will significantly reduce the fatigue stress ranges at details on or adjacent to the top flanges in regions of negative flexure or stress reversal. The concrete deck may also be considered effective in tension in such cases when calculating the stress due to the unfactored permanent loads applied to the composite section, i.e., DC_2 and DW loads, at the fatigue limit state (using the long-term modular ratio, *3n*, in this case to transform the concrete deck).

This Article also provides the criterion to determine if a component or detail is subject to a net tensile stress and therefore must be checked for fatigue. This criterion is applicable to components and details in routine steel multi-span continuous I-girder bridges located in regions where the unfactored permanent loads produce compression. In such cases, fatigue is to be checked only if the factored tensile stress in the component or at the detail due to the envelope of the fatigue live-load moments (factored for the Fatigue I load combination specified in Table 3.4.1-1; i.e., with a load factor of 1.75 applied to the fatigue live-load moment) exceeds the unfactored permanent load compressive stress in the component or at the detail. Of course, this discussion does not apply for simple span routine steel I-girder bridges as they are only subject to positive moments.

For further information on fatigue design, consult the NSBA's Steel Bridge Design Handbook – Chapter 12: Design for Fatigue, the Reference Manual for NHI Course 130122, Design and Evaluation of Steel Bridges for Fatigue and Fracture, and Section 6.5.5 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples illustrating load-induced fatigue design computations, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous

Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

Note that many commercial steel bridge design programs, such as NSBA's LRFD Simon line-girder analysis and design program, can perform the fatigue evaluation of the details on the girders of a routine steel I-girder bridge subject to a net tensile stress as specified in this Article. These programs will calculate the appropriate fatigue stress range in the girder flanges at these details for either the Fatigue I or Fatigue II limit-state load combination, as applicable, and compare it to the nominal fatigue resistance of the detail, determined for either an infinite life or a finite life evaluation as appropriate, based on parameters such as the detail category and the Average Daily Truck Traffic in a single lane in one direction, $(ADTT)_{SL}$, input by the user. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.6.1.2.2 Design Criteria

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article provides the basic equation for checking load-induced fatigue for components and details in steel I-girder bridges subject to a net tensile stress as specified in Article 6.6.1.2.1 (see the Discussion of Article 6.6.1.2.1 in this Guide); i.e., the live-load stress range due to the passage of the fatigue live load in a single lane (see the Discussion of Article 3.6.1.4.1 in this Guide), factored for either the Fatigue I or Fatigue II load combination specified in Table 3.4.1-1, as applicable, must not exceed the nominal fatigue resistance of the component or detail (see the Discussion of Article 6.6.1.2.5 in this Guide).

The discussion in the Commentary for this Article related to the checking of fatigue in cross-frames or diaphragms using the force effects computed from a refined analysis does not apply to the routine steel I-girder bridges covered by this Guide. Live-load force effects in cross-frame or diaphragm members are not available from the line girder analysis methods, which are normally used for the design of routine steel I-girder bridges. Designers need not be concerned about performing a fatigue analysis of cross-frame or diaphragm members in routine steel I-girder bridges; research has shown that, due to the nature of the geometry of the framing plan and overall layout of routine steel I-girder bridges, the live load force effects (and the resulting live load stress ranges) in the cross-frames or diaphragms of these bridges are typically not significant.

For further information on fatigue design, consult the NSBA's Steel Bridge Design Handbook – Chapter 12: Design for Fatigue, the Reference Manual for NHI Course 130122, Design and Evaluation of Steel Bridges for Fatigue and Fracture, and Section 6.5.5 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge

Superstructures. The reader is cautioned that the Reference Manuals for NHI Course 130122 and 130081 have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; these Reference Manuals still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.6.1.2.3 Detail Categories

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article provides an important table that defines the specific Detail Category and the value of the detail category constant, *A*, and constant-amplitude fatigue threshold, $(\Delta F)_{TH}$, for the load-induced fatigue design of typical components and details that may be encountered on steel bridges (Table 6.6.1.2.3-1). Furthermore, the table was revised in the 10th Edition of the AASHTO LRFD BDS to include the fatigue growth constant, *m*, (which was set at a fixed value of 3 for all details in earlier editions of the specifications, but is now presented as a variable whose value is based on the specific detail in question), the 75-year $(ADTT)_{SL}$ values that correspond to infinite fatigue life (which were presented in separate tables in earlier editions of the specification), and surface roughness value limits for Conditions 1.3 through 1.6, 3.3, and 8.7 (which were addressed by reference to the AWS specifications in earlier editions of the specifications). In addition, two new Detail Categories, 9.2 and 9.3, were added to the table in the 10th Edition, and the previous Detail Category 9.2 (nonpretensioned high-strength bolts, common bolts, threaded anchor rods, and hanger rods with cut, ground, or rolled threads subject to axial tension) was renumbered as Detail Category 9.4. The new Detail Categories 9.2 and 9.3 address shear connectors or base metal at shear connectors attached by fillet or automatic study welding (for use in fatigue design of shear connectors) and pretensioned high-strength bolts under axial tension, respectively. These revisions to Table 6.6.1.2.3-1 also provided the opportunity in the 10th Edition of the AASHTO LRFD BDS to eliminate Tables 6.6.1.2.3-2, 6.6.1.2.5-1, and 6.6.1.2.5-3, which appeared in earlier editions of the specifications but contained information that is now addressed in Table 6.6.1.2.3-1 of the specifications.

For the routine steel I-girder bridges covered by this Guide, the most common fatigue details that must be examined as a minimum when subject to a net tensile stress as specified in Article 6.6.1.2.1 (see the Discussion of Article 6.6.1.2.1 in this Guide) are the base metal away from welds and connections (Condition 1.1 in the table – Category A for all steels except for uncoated weathering steel; Condition 1.2 – Category B for uncoated weathering steel); base metal at the cross-section and net section of high-strength bolted joints (Conditions 2.1 and 2.2 – Category B); base metal adjacent to continuous flange-to-web fillet welds (Condition 3.1 – Category B); the base metal at the toe of transverse stiffener-to-flange fillet welds and transverse stiffener-to-web fillet welds, including bearing stiffener and cross-frame or diaphragm connection plate welds (Condition 4.1 if the stiffener is oriented normal to the longitudinal axis of the girder – Category C', or Condition 7.3 if the stiffener is obliquely oriented – Category C′, C, D, or E depending on the orientation of the stiffener); the base metal adjacent to complete joint penetration groove-welded flange splices (Condition 5.1 – Category B); the base metal at stud-type shear connectors attached to the top flange by welds (Condition 9.1 – Category C); and shear connectors or base metal at shear connectors attached by fillet or automatic stud welding (Condition 9.2 – which does not list an applicable Detail Category but instead provides information directly used in the fatigue design of shear connectors). Other conditions in the table typically do not apply to routine steel I-girder bridges, including Conditions 1.3 through 1.6, 2.3, 2.4, 3.2 through 3.7, 4.3, 4.4, 5.3 through 5.4, 6.1 through 6.4, and 7.1, which address details that typically are not (and should not be) used in the routine steel I girder bridges covered by this Guide and Conditions 8.1 through 8.9 covering orthotropic deck details. Condition 4.2 for base metal at the toe of half-round I-girder bearing stiffener-to-flange fillet welds and half-round I-girder bearing stiffener-to-web welds was introduced in the 10th Edition of the AASHTO LRFD BDS; the use of this detail is typically reserved for cases of fairly severe skew but may be considered on some moderately skewed steel I-girder bridges. See the Discussion of Article 6.10.11.2 and associated sub-articles for more guidance on the use of half-round bearing stiffeners.

In routine multi-span continuous I-girder bridges, the Category C' fatigue check of the connection plate-to-bottom flange weld (Condition 4.1) may (and in many cases does) control the size of the bottom flange in regions of positive flexure. If the nominal fatigue resistance is exceeded, the most common design change involves increasing the size of the bottom flange, although in some cases moving the connection plate to a slightly different location may suffice to address a situation where there is a minor exceedance of the nominal fatigue resistance; bolting the connection plate to the flange to raise the fatigue category to Category B (Condition 2.1) is not recommended as this detail is typically more expensive to fabricate and there is a fatigue Category C' detail at the termination of the connection plate-to-web weld only a short distance above the flange.

Fatigue does not typically control for bolted field splices. The nominal fatigue resistance of base metal at the gross section adjacent to slip-critical bolted connections with the bolts installed in holes drilled full size or subpunched and reamed to size is Category B (Condition 2.1); the combined areas of the flange and web splice plates must equal or exceed the areas of the smaller flanges and web to which they are attached, and the flanges and web are usually checked separately for either equivalent or more critical fatigue category details.

The welded connections typically used to attach angle or tee section (WT) cross-frame members to gusset plates in the truss-type cross-frames used in many steel I-girder bridges are designated as Category E' details (Condition 7.2), and as such have very low fatigue resistance. Bolted connections of cross-frame members to gusset plates, which are less commonly used and are not recommended, are typically punched full size by the fabricator and are designated as Category D details (Conditions 2.3 and 2.5). Where permitted for use, this Article states that unless specific information is available to the contrary, bolt holes in bracing members and their connection plates are to be assumed for design to be punched full size. However, as explained in the Discussion of Article 6.6.1.2.1 in this Guide, designers need not be concerned about performing a fatigue analysis of cross-frame or diaphragm members in routine steel I-girder bridges; that is, due to the nature of the geometry of the framing plan and overall layout of routine steel I-girder bridges, the live load force effects (and the resulting live load stress ranges) in the cross-frames or diaphragms of these bridges are typically not significant (even for Category E′ details).

This Article also provides the 75-year Average Daily Truck Traffic in a single lane in one direction, $(ADTT)_{SL}$, Equivalent to Infinite Life for each fatigue-detail category (specified in Table 6.6.1.2.3-

1 for each fatigue-detail detail category). The 75-year $(ADTT)_{SL}$ may be calculated as the fraction of traffic in a single lane, *p*, given in Table 3.6.1.4.2-1 times the *ADTT* in one direction averaged over the design life. Article C3.6.1.4.2 provides helpful recommendations on how to estimate the 75-year *ADTT* in one direction (see the Discussion of Article 3.6.1.4.2 in this Guide). For the component or detail under consideration, if the calculated 75-year $(ADTT)_{SL}$ in a single lane in one direction exceeds the value of the 75-year $(ADTT)_{SL}$ Equivalent to Infinite Life specified in Table 6.6.1.2.3-1 for the corresponding fatigue-detail category, that component or detail is to be designed for infinite life using the Fatigue I load combination in Table 3.4.1-1. Otherwise, the component or detail is to be designed for finite life using the Fatigue II load combination (see the Discussion of Article 6.6.1.2.5 in this Guide). In such cases, details on the routine steel I-girder bridges covered by this Guide should not be designed for infinite life, unless required to do so by Owner-agency policy. The values given in Table 6.6.1.2.3-1 are calculated from Eq. C6.6.1.2.3-1. For a number of stress range cycles per truck passage, *n*, other than 1.0, the values in Table 6.6.1.2.3-1 should be modified by dividing the values by the appropriate value of *n* taken from Table 6.6.1.2.5-1. For values of the fatigue design life other than 75 years, the values in Table 6.6.1.2.3-1 should be modified by multiplying the values by the ratio of 75 divided by the fatigue life sought in years.

The remaining provisions in this Article dealing with nonredundant steel tension members (NSTMs) and orthotropic deck components and details are not applicable to the routine steel I-girder bridges covered by this Guide.

For further information on fatigue design, consult the NSBA's Steel Bridge Design Handbook – Chapter 12: Design for Fatigue, the Reference Manual for NHI Course 130122, Design and Evaluation of Steel Bridges for Fatigue and Fracture, and Section 6.5.5 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manuals for NHI Course 130122 and 130081 have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; these Reference Manuals still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

Consult the AASHTO-NSBA Steel Bridge Collaboration Guidelines G1.4-2006 Guidelines for Design Details and G12.1-2020 Guidelines to Design for Constructability and Fabrication for recommendations, commentary, and sample design details that allow for more economical fabrication and erection.

Note that many commercial steel bridge design programs, such as NSBA's LRFD Simon line-girder analysis and design program, can perform the fatigue evaluation of the details on the girders of a routine steel I-girder bridge subject to a net tensile stress as specified in Article 6.6.1.2.1. These programs will calculate the appropriate fatigue stress range in the girder flanges for either the Fatigue I or Fatigue II limit-state load combination, as applicable, and compare it to the nominal fatigue resistance of the detail, determined for either an infinite life or a finite life evaluation as appropriate, based on parameters such as the detail category and the Average Daily Truck Traffic in a single lane in one direction, $(ADTT)_{SL}$, input by the user. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.6.1.2.4 Detailing to Reduce Constraint

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides recommended guidelines for specific details involving intersecting welded elements to avoid highly constrained joints and crack-like geometric discontinuities that could potentially be susceptible to constraint-induced fracture (CIF). Routine steel I-girder bridges covered by this Guide do not typically contain such details (e.g., longitudinal web stiffeners intersecting transverse web stiffeners or lateral connection plates intersecting transverse web stiffeners), and so the provisions of this Article are not applicable to the routine steel I-girder bridges covered by this Guide.

6.6.1.2.5 Fatigue Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article provides the provisions necessary to compute the nominal fatigue resistance, $(\Delta F)_n$, of components or details subject to a net tensile stress as specified in Article 6.6.1.2.1 (see the Discussion of Article 6.6.1.2.1 in this Guide) in steel I-girder bridges.

For components and details to be designed for infinite life under the Fatigue I load combination, which is representative of the maximum stress range of the truck population, the nominal fatigue resistance is simply taken equal to the constant-amplitude fatigue threshold, $(\Delta F)_{TH}$, which is given for each fatigue-detail category in Table 6.6.1.2.3-1. This occurs whenever the calculated 75-year $(ADTT)_{SL}$ exceeds the value of the 75-year $(ADTT)_{SL}$ Equivalent to Infinite Life specified in Table 6.6.1.2.3-1 for the applicable fatigue-detail category (see the Discussion of Article 6.6.1.2.3 in this Guide).

For components and details to be designed for finite life under the Fatigue II load combination, which is representative of the effective stress range of the truck population, the nominal fatigue resistance is taken equal to $(A/N)^{1/m}$, where A is the y-intercept of the sloping portion of the *S-N* curve given for each fatigue-detail category as specified in Table 6.6.1.2.3-1, N is the number of stress range cycles given by Eq. 6.6.1.2.5-3, and m is the fatigue growth constant (also found in Table 6.6.1.2.3-1) corresponding to the slope of the S-N curve in the finite-life region. In earlier editions of the AASHTO LRFD BDS, the fatigue growth constant was set at a fixed value of 3 for all detail categories, but in the 10th Edition a generic variable, m, was introduced to allow the use of different values of the slope of the S-N curve based on the detail. All details except shear connectors are assigned a value of $m = 3$, while shear connectors are assigned a value of $m = 5$. The slope of the S/N curve in log-log format for shear connectors is based on a more recent regression analysis of historical shear connector fatigue test data.

Design for finite fatigue life is allowed whenever the calculated 75-year $(ADTT)_{SL}$ is less than or equal to the value of the 75-year $(ADTT)_{SL}$ Equivalent to Infinite Life specified in Table 6.6.1.2.3-1 for the applicable fatigue-detail category (see the Discussion of Article 6.6.1.2.3 in this Guide). In such cases, details on the routine steel I-girder bridges covered by this Guide should be designed for finite life rather than infinite life, unless otherwise required by Owner-agency policy. If a fatigue design life other than 75 years is sought, a number other than 75 may be inserted in the equation for N. The necessary adjustments to the tabulated values given in Table 6.6.1.2.3-1 for a fatigue design life other than 75 years are discussed in Article C6.6.1.2.3 (see the Discussion of Article 6.6.1.2.3 in this Guide). Values of A and $(\Delta F)_{TH}$ specified for bolts subject to axial tension in Tables 6.6.1.2.3-1 are typically not applicable to routine steel I-girder bridges.

Eq. 6.6.1.2.5-4 for calculating the nominal fatigue resistance of base metal at details where loaded discontinuous plate elements are connected with a pair of fillet welds or partial joint penetration groove welds on opposite sides of the plate normal to the direction of primary stress, or where partial joint penetration groove welds are transversely loaded in tension, is not applicable to the routine steel I-girder bridges covered by this Guide. These types of details, including details utilizing partial joint penetration groove welds, are not typically used in these bridges.

For simple span bridges, the number of stress cycles per truck passage, n, used in the computation of N in Eq. 6.6.1.2.5-3 is taken equal to 1.0 (Table 6.6.1.2.5-1). For multi-span continuous steel plate girder or rolled-beam bridges, the number of stress cycles per truck passage, n, used in the computation of N in Eq. 6.6.1.2.5-3 is taken equal to 1.5 for details located within one-tenth of the span length on either side of an interior support; otherwise, n is taken equal to 1.0 (Table 6.6.1.2.5-1). The other values of n specified in Table 6.6.1.2.5-1 are not applicable to the routine steel I-girder bridges covered by this Guide. The necessary adjustments to the tabulated values of the 75-year $(ADTT)_{SL}$ Equivalent to Infinite Life given in Table 6.6.1.2.3-1 for a value of n other than 1.0 are discussed in Article C6.6.1.2.3 (see the Discussion of Article 6.6.1.2.3 in this Guide).

The provisions and commentary related to the number of stress-range cycles per truck passage for cross-frames and diaphragms are not applicable to the design of the routine steel I-girder bridges that are the subject of this Guide. As noted elsewhere (see the Discussion of Article 6.6.1.2.2), designers need not be concerned about performing a fatigue analysis of cross-frame or diaphragm members in routine steel I-girder bridges; research has shown that, due to the nature of the geometry of the framing plan and overall layout of routine steel I-girder bridges, the live load force effects (and the resulting live load stress ranges) in the cross-frames or diaphragms of these bridges are typically not significant.

For further information on fatigue design, consult the NSBA's Steel Bridge Design Handbook – Chapter 12: Design for Fatigue, the Reference Manual for NHI Course 130122, Design and Evaluation of Steel Bridges for Fatigue and Fracture, and Section 6.5.5 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples illustrating load-induced fatigue design computations, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

Note that many commercial steel bridge design programs, such as NSBA's LRFD Simon line-girder analysis and design program, can perform the fatigue evaluation of the details on the girders of a routine steel I-girder bridge subject to a net tensile stress as specified in Article 6.6.1.2.1. These programs will calculate the appropriate fatigue stress range in the girder flanges for either the Fatigue I or Fatigue II limit-state load combination, as applicable, for comparison to the nominal fatigue resistance of the detail, determined for either an infinite life or a finite life evaluation as appropriate, based on parameters such as the detail category and the Average Daily Truck Traffic in a single lane in one direction, $(ADTT)_{SL}$, input by the user. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.6.1.3 Distortion-Induced Fatigue

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The provisions of this Article are applicable to the routine steel I-girder bridges covered by this Guide to check that rigid load paths are provided to adequately transmit the forces in transverse bracing members from the beam or girder web to the flanges in order to prevent distortion-induced fatigue, which is defined as fatigue effects due to secondary out-of-plane stresses not normally quantified in the typical analysis and design of a bridge. The rigid load paths are provided by attaching the various components through either welding or bolting.

The provision of Article 6.10.5.3 mentioned in this Article is applicable to the routine steel I-girder bridges covered by this Guide (see the Discussion of Article 6.10.5.3 in this Guide).

6.6.1.3.1 Transverse Connection Plates

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

The provisions of this Article relate to the detailing necessary to prevent distortion-induced fatigue from occurring at transverse connection plate details (i.e., vertical stiffeners attached to a beam or girder to which a cross-frame or diaphragm is attached, including bearing stiffeners).

Except for the special case of straight, rolled-beam bridges with supports skewed less than or equal to 10 degrees from normal as permitted later in this Article, the first bulleted item in this Article requiring transverse connection plates to be positively attached (by welding or bolting) to the compression and tension flanges of the beam or girder applies to the routine steel I-girder bridges covered by this Guide. Welded connections are preferred as bolted connections possessing sufficient stiffness are not likely to be economical. The connections for routine steel I-girder bridges should be designed as a minimum for the larger of the calculated resultant force in those members or the factored 20-kip lateral load specified in this Article for straight, nonskewed bridges.

The second and third bullets in this Article are not applicable to the routine steel I-girder bridges covered by this Guide as such bridges do not contain internal or external cross-frames or diaphragms, floor beams, or stringers.

For the for the special case of straight, rolled-beam bridges with composite reinforced decks, whose supports are skewed less than or equal to 10 degrees from normal, with the diaphragms placed in contiguous lines parallel to the supports, an option is provided to allow the use of less than full-depth end angles or connection plates bolted or welded to the web if permitted by the Owner; the end angles or plates must satisfy the requirements specified in this Article.

Consult the AASHTO-NSBA Steel Bridge Collaboration Guidelines G1.4-2006 Guidelines for Design Details and G12.1-2020 Guidelines to Design for Constructability and Fabrication for recommendations, commentary, and sample design details that allow for more economical fabrication and erection.

6.6.1.3.2 Lateral Connection Plates

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article relate to the detailing necessary to prevent distortion-induced fatigue from occurring at lateral connection plate details (i.e., plates used to interconnect lateral bracing members for attachment to a flexural member). These provisions are not used in most of the routine steel I-girder bridges covered by this Guide, which generally do not utilize any lateral bracing.

However, in unusual situations the use of a limited amount of top flange lateral bracing near the ends of the span may represent an effective and practical way to improve stability or control horizontal wind deflections. For instance, moderately or extremely narrow single span routine steel I-girder bridges with span lengths in excess of approximately 160 feet may exhibit insufficient stability during construction; the use of top flange lateral bracing is one possible solution (see the Discussion of Article 6.7.4.2.2 about stability bracing requirements for steel I-girder bridges). Similarly, single span bridges with span lengths near the upper end of the 200-foot span length limit of the routine steel I-girder bridges considered in this Guide may exhibit excessive horizontal displacement under wind loading during construction; the use of top flange lateral bracing is one possible solution (see the Discussion of Article 3.4.2.2 about evaluation of deflections at the service limit state).

6.6.1.3.3 Orthotropic Decks

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article relate to the detailing necessary to prevent distortion-induced fatigue from occurring at orthotropic deck details. These provisions are not applicable to the routine steel I-girder bridges covered by this Guide, which are assumed to have only concrete decks.

6.6.2 Fracture

6.6.2.1 Member or Component Designations and Charpy V-Notch Testing Requirements

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article provides member or component designations (i.e., primary or secondary) and Charpy V-notch testing requirements to evaluate the fracture toughness of the steel for the applicable temperature zone. These provisions are applicable to the routine steel I-girder bridges covered by this Guide.

Beams and girders, including the splice plates for bolted field splices, are designated as primary members. Cross-frame members or diaphragms, cross-frame gusset plates, transverse connection plates, transverse intermediate web stiffeners, bearing stiffeners, and any nonstructural components or attachments (e.g., expansion dams, drainage material, brackets, etc.) in routine steel I-girder bridges are designated as secondary members (Table 6.6.2.1-1). Arbitrary or "conservative" designation of secondary members or components as primary members or components is discouraged, as this will invoke more costly and complex fabrication and testing requirements that do not add significant value and are not necessary. See also the Discussions of Articles 6.8.4 and 6.9.3 for situations when cross-frame members may be considered as "primary members" when evaluating slenderness ratios. Designations for bracing members in composite box-girder bridges in Table 6.6.2.1-1 are not applicable to the routine steel I-girder bridges covered by the Guide.

Primary members or components, or portions thereof, subject to a net tensile stress under the Strength I limit state (Table 3.4.1-1) must be designated on the contract plans. Charpy V-notch testing is required for primary members or components subject to a net tensile stress, or for portions thereof located in designated tension zones, under Strength I. The testing is done by the steel producers.

Specifying that Charpy V-notch testing be performed for secondary members (e.g., cross-frame or diaphragm members) adds additional complexity and cost without providing any significant additional value and should not be done for routine steel-girder bridges.

The minimum Charpy V-notch toughness requirements for various grades of steel for the three temperature zones specified in Table 6.6.2.1-2 are given in Table C6.6.2.1-1. The toughness requirements in Table C6.6.2.1-1 for Members or Designated Tension Zones Requiring FC Practice and for Grades HPS 50W, HPS 70W, and HPS 100W are not applicable for the routine steel I-girder bridges covered by this Guide.

Generally, Owner-agency policy will establish the minimum service temperature used to determine the Temperature Zone in Table 6.6.2.1-2.

6.6.2.2 NSTMs

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article contains provisions for nonredundant steel tension members (NSTMs), which are defined as steel primary members fully or partially in tension, and without load path redundancy, system redundancy or internal redundancy, whose failure may cause a portion of or the entire bridge to collapse. The routine steel I-girder bridges covered by this Guide do not contain NSTMs. A routine steel I-girder bridge, as defined in this Guide, features a cross-section with four or more girders and a composite concrete deck, and so the bridge is considered load path redundant; i.e., the bridge provides multiple redundant load paths for gravity loads (dead loads and live loads). As such, the provisions of this Article are not applicable.

6.7 GENERAL DIMENSION AND DETAIL REQUIREMENTS

6.7.1 Effective Length of Span

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article provides the definition of the effective length of a span, which is typically measured between bearing locations and is applicable to the routine steel I-girder bridges covered by this Guide.

6.7.2 Dead Load Camber and Detailing of Structural Components

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article contains provisions related to determining structure dead load deflections and cambers and specifying them in contract documents. For routine steel I-girder bridges, deflections accounting for the self-weight of the steel, the weight of the wet concrete deck on the noncomposite section, and additional dead loads acting on the composite section (such as the weight of barriers or a future wearing surface), should be determined separately, and typically can be calculated using the same line-girder analysis model used to calculate girder force effects (such as NSBA's LRFD Simon line-girder analysis and design program). The specific method for specifying vertical cambers on the contract drawings is often dictated by Owner-agency requirements.

This Article also discusses the considerations for reporting dead load deflections and vertical cambers when staged deck placement or phased construction is specified. In the context of the AASHTO LRFD BDS, the term "staged deck placement" refers to the placement of the deck in discrete pours that are the full width of the bridge deck but only part of the length of the bridge. The pours should be placed on the deck in a specific sequence identified to minimize tensile stresses in the deck, typically by placing pours in the positive moment regions first, and negative moment regions later. In the context of the AASHTO LRFD BDS, the term "phased construction" refers to building the bridge in partial-width phases, where one phase is fully constructed (i.e., both the steel superstructure and the deck are constructed), prior to the construction of the next phase. Bridge widenings are conceptually similar to phased construction and behave similarly.

For routine steel I-girder bridges, line-girder analysis software (such as NSBA's LRFD Simon line-girder analysis and design program) can typically accommodate the analysis for staged deck placement (or pours) along the length of the structure (see the Discussion of Article 6.10.3.4 in this Guide).

For routine steel I-girder bridges as defined in this Guide, it is assumed that only single-phase construction, simple multi-phase construction, or simple bridge widenings are being considered (see the Definition of a "Routine Steel I-Girder Bridge"), so as to maintain the applicability of a line-girder analysis. For more complicated situations, such as cases where a closure pour is not provided between adjacent phases of construction, the use of a refined analysis method may be warranted for proper calculation of load distribution, stresses, and deflections. Refer to the AASHTO-NSBA Steel Bridge Collaboration's Guideline G13.1-2019 Guidelines for Steel Girder Bridge Analysis and to Sections 6.3.2.5.4 and 6.5.3.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures for further guidance on staged deck placement and phased construction considerations.

This Article also presents requirements related to changes in component length to account for cambering of structures such as trusses, arches, and cable-stayed systems. These requirements do not apply to the routine steel I-girder bridges covered by this Guide.

Finally, this Article requires the identification of a "fit condition" in the contract documents for certain specific types of I-girder bridges. The identification of the desired fit condition of an I-girder bridge refers to the identification of the targeted girder dead load geometry condition (no load, steel dead load, or total dead load) for which the cross-frames or diaphragms are detailed to connect to the girders. The Fabricator/Detailer determines the cross-frame or diaphragm geometry based on the vertical deflections provided in the contract documents and the specified fit condition. I-girder bridges requiring the specification of a fit condition include the following:

- straight bridges where one or more support lines are skewed more than 20 degrees from normal;
- horizontally-curved bridges where one or more support lines are skewed more than 20 degrees from normal and with an L/R in all spans less than or equal to 0.03; and
- horizontally-curved bridges with or without skewed supports and with a maximum L/R greater than 0.03.

For the purposes of this Guide, routine steel I-girder bridges do not meet any of these criteria and the requirement to identify a desired fit condition, along with requirements to consider the effect of the selected fit condition on bearing rotations mentioned in the last two paragraphs of this Article and extensively discussed in the Commentary, do not apply.

For interested readers, the NSBA has published both a brief guide on steel I-girder bridge fit, Skewed and Curved Steel I-Girder Bridge Fit (Executive Summary), and a longer, more in-depth, white paper on the topic, Skewed and Curved Steel I-Girder Bridge Fit (Full White Paper).

6.7.3 Minimum Thickness of Steel

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article specifies the minimum thickness (0.3125 in.) for structural steel, such as girder webs and flanges, bracing members (i.e., cross-frames or diaphragm members), stiffeners, and gusset plates/connection plates for cross-frames or diaphragms. Filler plates, webs of rolled shapes, and material in barrier railings are exempt from this requirement. The web thickness of rolled beams or channels is not to be less than 0.23 in. The AASHTO-NSBA Steel Bridge Collaboration Guideline G12.1-2020 Guidelines to Design for Constructability and Fabrication suggests practical values of the minimum thickness of many of these components, which typically significantly exceed the absolute minimum thickness value specified herein.

The additional requirements in this Article for truss gusset plates and orthotropic decks do not apply to the routine steel I-girder bridges covered by this Guide.

Consult Owner-agency policy regarding the need to provide special protection against corrosion or to specify a sacrificial metal thickness to account for potential section loss due to corrosion.

6.7.4 Diaphragms and Cross-Frames

6.7.4.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article covers spacing and detailing requirements and other general design requirements for cross-frames or diaphragms in various steel structure types. In the AASHTO LRFD BDS, a cross-frame is defined as a transverse truss framework connecting adjacent longitudinal flexural components or inside a tub section or closed box used to transfer and distribute vertical and lateral loads and to provide stability to the compression flanges. A diaphragm is defined as a vertically oriented solid transverse member connecting adjacent longitudinal flexural components or inside a closed-box or tub section to transfer and distribute vertical and lateral loads and to provide stability to the compression flanges.

Diaphragms or cross-frames should be placed at end and interior supports, with the spacing of intermediate diaphragms or cross-frames based on an investigation of the specified stages of construction and the final condition. The diaphragms or cross-frames should be designed with sufficient stiffness to function as brace points for the girders and with sufficient strength to resist their anticipated loading, including consideration of dead loads, live loads, wind loads, construction loads, and stability bracing forces, as appropriate. At a minimum, the design should consider:

- transfer of wind loads from the bottom of the girder to the deck and from the deck into the support bearings;
- lateral loading of flanges due to overhang falsework during deck placement;

- providing bracing with sufficient strength and stiffness to function as brace points that contribute to the lateral torsional buckling resistance of the girders in regions of negative flexure at the strength limit state and in both regions of positive and negative flexure during critical stages of construction;
- limiting flange lateral bending moments to reasonable levels; and
- the distribution of applied dead and live loads across the width of the structure.

The Article specifically prohibits consideration of the contribution of metal stay-in-place deck forms to the stability of the top flange of the noncomposite member in compression.

For the routine steel I-girder bridges covered by this Guide, i.e., bridges without curvature or significant skew, the spacing of cross-frames or diaphragms is dictated primarily by the need to limit flange lateral bending stresses due to overhang bracket or wind loads and to provide an appropriate unbraced length to control lateral-torsional buckling.

As stated in the Commentary for this Article, the AASHTO 25.0-ft spacing limit on intermediate cross-frames or diaphragms that had existed in Specifications prior to the AASHTO LRFD BDS has been removed. However, for a routine I-girder bridge, this is still a reasonable upper limit to achieve reasonably sized flanges and bracing members and to control stresses in the concrete deck. Additionally, Owner-agency policy may dictate maximum spacing limits.

Designers may choose to vary the cross-frame or diaphragm spacing within each span, using tighter spacings near interior supports to reduce the unbraced length when the bottom flange is in compression. Variations in spacing should be kept to a minimum; using multiple different spacings within a given span in response to changes in the magnitude of girder moments is neither warranted nor recommended. During the layout of the framing plan, a cursory review of compression flange lateral-torsional buckling resistances in Article 6.10.8.2.3 or A6.3.3, as applicable, can be made to determine the sensitivity of the lateral-torsional buckling resistance to various unbraced lengths.

The requirements in this Article for including the bracing in the analysis model and considering the force effects of horizontal curvature do not apply to the routine steel I-girder bridges covered by this Guide. However, the requirement to design cross-frames or diaphragms to transfer wind loads and meet slenderness requirements, at a minimum, does apply. In addition, the 10th Edition of the AASHTO LRFD BDS introduced provisions in a new Article 6.7.4.2.2 requiring cross-frames or diaphragms also be designed to satisfy stability bracing strength and stiffness requirements. Similar provisions have been specified in AISC Appendix 6 - Article 6.3.2a for a number of years. Consult Section 6.6.3.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures, the NSBA's Steel Bridge Design Handbook – Chapter 13: Bracing System Design, and the Discussion of Article 6.7.4.2.2 in this Guide for further information on these requirements and their application to cross-frames and diaphragms in steel I-girder bridges. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

As noted elsewhere (see the Discussion of Article 6.6.1.2.2), designers need not be concerned about performing a fatigue analysis of cross-frame or diaphragm members in routine steel I-girder bridges; research has shown that, due to the nature of the geometry of the framing plan and overall layout of routine steel I-girder bridges, the live load force effects (and the resulting live load stress ranges) in the cross-frames or diaphragms of these bridges are typically not significant. In fact, as noted in the Commentary to this article (C6.7.4.1), the force demands in intermediate cross-frames and diaphragms not only for live load on the completed structure, but also other loading effects (such as dead load) are generally small in the routine steel I-girder bridges that are the subject of this Guide. As noted throughout this Guide, developing a refined analysis model for these types of routine steel I-girder bridges is neither warranted nor recommended; line girder analysis is the recommended method of analysis for the routine steel I-girder bridges that are the subject of this Guide.

Unless the cross-frame or diaphragm members are directly connected to the girder flanges, except as permitted in Article 6.6.1.3.1, the connection plates are to be attached directly to the beam or girder flanges, with the attachment designed for the larger of the calculated resultant force in those members or the recommended minimum factored 20.0-kip value specified in Article 6.6.1.3.1 (see the Discussion of Article 6.6.1.3.1 in this Guide). In addition to the wind loads, stability bracing strength and stiffness requirements, and slenderness requirements, the minimum factored 20.0-kip horizontal design force for the attachment between the cross-frame or diaphragm connection plates and the beam or girder flanges may be considered as a design limit for the cross-frame or diaphragm member(s) as well.

Finally, the Article requires that deck slab edges be supported per Article 9.4.4, whether by cross-frame top struts (typically rolled steel beam wide-flange or channel shapes), steel diaphragms, integral concrete diaphragms, or thickened slab ends. Typical transverse deck reinforcement is not sized to carry vehicular wheel loads at these discontinuous locations without additional support. The extra support also minimizes differential movement issues at expansion joint devices. Bridges with decks continuous over the backwall are typically not subject to this requirement. The Article also specifies similar supports anywhere the slab continuity is broken between the ends of the bridge, such as at hinge locations in older structures (a condition that is not applicable to the design of new routine steel I-girder bridges covered by this Guide).

For further information on cross-frames and diaphragms, consult Section 6.3.2.9 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures and various chapters of NSBA's Steel Bridge Design Handbook. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.7.4.2 I-Section Members

6.7.4.2.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article presents general design requirements for cross-frames or diaphragms in bridges containing steel I-section members.

The Article specifies the required minimum depth of cross-frames or diaphragms, as a function of the overall girder depth, dependent on the girder type (i.e., rolled beam or plate girder). This Article also invokes a requirement that diaphragms or cross-frames satisfy the stability bracing strength and stiffness requirements presented in Article 6.7.4.2.2 (see the Discussion of Article 6.7.4.2.2 in this Guide for more information). Additionally, end diaphragm design forces transmitted by the deck and deck joint are discussed as well as the transfer of these forces to the girders. The provisions of this Article permit solid web diaphragms to be designed by traditional beam theory when their span-to-depth ratio is greater than or equal to 4.0. Otherwise, solid web diaphragms should be treated as deep beams and designed for principal stresses. These requirements are applicable to all steel girder bridges.

For straight bridges with support skews less than or equal to 20 degrees (as is the case for the routine steel I-girder bridges discussed in this Guide), the applicable provisions of this Article allow intermediate cross-frames or diaphragms to be placed in contiguous lines parallel to the skewed support lines. This is recommended, since it simplifies the design, detailing, and fabrication of the bridge.

Other framing requirements for bridges with larger skew angles and for horizontally curved bridges are not applicable to the routine steel I-girder bridges covered by this Guide. The requirement to consider the effect of the tangential component of force in a skewed end cross-frame or diaphragm on the beam or girder is also considered not applicable, as this effect is not significant for the routine I-girder bridges covered by this Guide.

Finally, this Article requires that cross-frames or diaphragms at supports be designed to transmit lateral forces from the superstructure to the bearings that provide lateral restraint. These lateral forces primarily include the forces due to wind load, seismic, and centrifugal force effects. For the routine steel I-girder bridges covered by this Guide, only lateral forces due to wind-load effects need to be considered as these bridges are assumed located in Seismic Zone 1 and are not horizontally curved.

The NSBA's Steel Bridge Design Handbook, specifically NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, provides design examples for intermediate and end diaphragms. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.7.4.2.2 Stability Bracing Requirements

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article presents specific design provisions related to the minimum strength and stiffness required of cross-frames or diaphragms for them to adequately function as bracing for the girders in bridges with superstructures consisting of steel I-girders.

The girders in steel I-girder bridges (including the routine steel I-girder bridges covered by this Guide) are subject to lateral-torsional buckling under gravity loading (dead load and live load). Lateral-torsional buckling is an instability failure mode in which the girders twist and the compression flanges buckle laterally. The resistance of a single I-girder to lateral-torsional buckling is a function of the strength and the stiffness of the girder and its component elements, and the unbraced length of the girder. The unbraced length of the girder is the length – between points of bracing – over which the girder can twist, and its compression flange can displace laterally. As noted in the Commentary to Article 6.7.4.2.2, “Effective stability bracing can be achieved by either preventing lateral movement of the compression flange with lateral bracing or by controlling twist of the cross-section with torsional bracing.” Cross-frames and diaphragms are forms of *torsional bracing*; they connect adjacent girders to each other so that one girder cannot twist without engaging the vertical stiffness of one or more adjacent girders to resist that twist.

For a cross-frame or diaphragm to adequately function as a brace to resist lateral-torsional buckling of I-girders it must possess two primary features:

- sufficient stiffness to control the magnitude of the deformations of the I-girders; and,
- sufficient strength to resist the internal member forces that can develop in the diaphragms or cross-frames as they work to resist lateral-torsional buckling of the girders.

The stiffness of the cross-frames or diaphragms affects the magnitude of the deformations of the I-girders. During lateral-torsional buckling, the compression flange of the I-girder displaces laterally in a manner similar to the bowing of a column that begins to buckle when subjected to an axial compressive force. The cross-frame or diaphragm acts as a brace to restrict that lateral displacement of the compression flange; the stiffer the cross-frame or diaphragm is, the smaller the magnitude of the lateral displacement of the girder’s compression flange. Controlling this displacement is important for two reasons:

- The larger the lateral displacement of the flange, the greater the eccentricity of the compression force in the girder flange. The greater the eccentricity, the larger the second-order moment in the flange. The larger the moment, the greater the beam-column loading in the flange. If the cross-frame or diaphragm has stiffness equal to or greater than the so-called “ideal stiffness,” these second order effects will converge to a stable condition. If not, the second order effects will grow larger and larger until the flange buckles.
- Also, the larger the lateral displacement of the flange, the larger the forces will be in the cross-frame or diaphragm as it tries to resist the lateral-torsional buckling of the girder.

The AASHTO provisions are based on providing either two times or three times the ideal stiffness, depending on the depth of the cross-frame or diaphragm. For cross-frames or diaphragms whose depth is at least 80% of the girder depth, the stiffness requirement reflects providing at least two times the ideal stiffness; for shallower cross-frames or diaphragms, the stiffness requirement reflects providing at least three times the ideal stiffness.

The stiffness of the cross-frame or diaphragm is comprised of three components:

- The stiffness of the cross-frame or diaphragm itself, called the "brace stiffness" and denoted as β_{br}. This is a function of the size and geometric configuration of the various component parts of the cross-frame or diaphragm itself.

- The cross-sectional distortion stiffness for stability bracing, denoted as β_{sec}. This is a representation of how much the girder cross-section will distort. If the cross-frame or diaphragm is nearly full depth (specified as at least 80% of the depth of the girder), the potential distortion of the cross-section will be essentially negligible, and cross-sectional distortion stiffness can be taken as infinity. If, on the other hand, the depth of the cross-frame or diaphragm is less than 80% of the girder depth, an equation is provided to calculate β_{sec}.

- The in-plane girder stiffness, denoted as β_g. This is a representation of the stiffness of the girders to which the cross-frames or diaphragms are attached and is a function of the vertical bending stiffness of the girders, the number of girders, the spacing between the girders, and the span length of the girders.

The overall provided stiffness of the torsional bracing system (of which the cross-frames or diaphragms are one part) is calculated as the inverse of the sum of the flexibilities of each of these three components (see Eq. 6.7.4.2.2-6). These three components work together in a manner similar to springs in series; the overall stiffness of the torsional bracing system is less than the individual stiffness of the least stiff of the three components of that system. While the actual interrelated behavior of the steel superstructure system of girders and cross-frames is complex, the equations to calculate β_{br}, β_{sec}, and β_g presented in this Article are quite simple, as they are comprised of straightforward terms representing basic structural parameters that are easily identified and quantified. In this way, these equations provide an easy way to approximate the performance of the system.

For most cases of the routine steel I-girder bridges covered by this Guide, the cross-frames or diaphragms will be essentially full-depth or nearly so, such that the cross-sectional distortion stiffness term, β_{sec}, can be taken as infinity and will drop out of Eq. 6.7.4.2.2-6.

The brace stiffness, β_{br}, is easy to determine. Equations are provided directly in the Article for calculating the brace stiffness, β_{br}, for a variety of common cross-frame and diaphragm configurations. Generally, a cross-frame or diaphragm with component members sized similarly to past designs will be sufficient to satisfy the design requirements.

The in-plane girder stiffness, β_g, warrants further discussion. The in-plane girder stiffness is strongly influenced by the span length, L. As can be seen in Eq. 6.7.4.2.2-13, β_g is a function of the inverse of L^3 and thus the in-plane girder stiffness decreases exponentially in relation to increases in the span length. Meanwhile, β_g is only linearly related to the value of vertical bending stiffness of the girder, EI_x. Consequently, it can take a significant increase in the size and stiffness of the girders to overcome even a moderate increase in the span length. Studies of the bracing stiffness requirements for routine steel I-girder bridges have suggested that at longer span lengths (in the range of roughly 160 to 200 feet for single span structures, for example), the in-plane girder stiffness (β_g) term dominates the overall torsional bracing system stiffness and an acceptable design cannot be achieved with reasonably sized girders and cross-frames or diaphragms. The recommended approach in this situation is not to try to increase the in-plane girder stiffness term by increasing the size and stiffness of the girders themselves. A much more effective solution is to provide flange-level lateral bracing (preferably top flange lateral bracing) or hardened placement of the concrete deck extending over a portion of the length of the span adjacent to each support. This develops the equivalent of a fixed-end condition for global lateral-torsional buckling, which corresponds to effectively shortening the equivalent length of the span. In such situations, Article 6.7.4.2.2 allows for a reduction in the required stiffness of the torsional brace system, $(\beta_T)_{req.}$, and the Commentary provides helpful explanations and suggestions for how a designer might deal with this situation. In rare cases, it may be appropriate to conduct a more refined analysis, such as an elastic buckling analysis (using a three-dimensional shell-element finite element model that captures the significant aspects of stiffness and geometry of the superstructure system), to evaluate the global lateral-torsional buckling stability of the bridge. However, such analyses are complex, time-consuming to perform, and are generally not warranted for the routine steel I-girder bridges that are the subject of this Guide.

In addition to needing to provide sufficient stiffness, cross-frames and diaphragms also need to provide sufficient strength to resist the internal member forces that can develop in the cross-frames or diaphragms as they work to resist lateral-torsional buckling of the girders. This Article provides two equations for estimating the required strength of a torsional brace (i.e., the required strength of a cross-frame or diaphragm). Eq. 6.7.4.2.2-14 is used when the depth of the cross-frame or diaphragm is at least 0.8 times the beam or girder depth, and Eq. 6.7.4.2.2-15 is used otherwise. These equations parallel the equations for the required stiffness of a cross-frame or diaphragm (Eq. 6.7.4.2.2-2 and -3) and are basically the product of those equations multiplied by an assumed initial twist imperfection in the girders. The assumed critical imperfection is based on a lateral sweep of the compression flange equal to the unbraced length of the girder, L_b, divided by 500, which is then converted into a twist rotation by dividing the sweep by the girder depth, represented by the distance between the flange centroids, h_o. The resulting force demand on the torsional brace, M_{br}, is a moment demand on the cross-frame or diaphragm. This brace moment, M_{br}, can be used directly as the in-plane moment demand for the design of cross-frames or diaphragms. For the case of truss-type cross-frames, the brace moment, M_{br}, should be converted into individual member design forces in the cross-frame chord and diagonal members. Handy equations are provided in the Commentary in Figure C6.7.4.2.2-1 for the calculation of these individual member design forces as a function of the brace moment, M_{br}, along with associated Commentary explaining how the equations should be used to appropriately design the members. Since the brace moment, M_{br}, is calculated using the factored beam or girder moment, M_u, it is not necessary to apply a load factor to M_{br}.

In addition to considering the brace moment, M_{br}, the engineer should also account for other concurrent loading effects, as appropriate. For example, cross-frames and diaphragms are typically also designed to carry wind loads, overhang falsework loads, and other loads that may be induced in them. These concurrent loading effects are generally easy to determine using hand calculations based on simplifying assumptions. The effects of the brace moment, M_{br}, should be combined with these other concurrent loading effects, using appropriate load combinations for the various applicable limit states, to determine the controlling design loads for the cross-frame or diaphragm.

In addition to checking that the diaphragm or the cross-frame members are sufficiently sized to meet these strength requirements, the engineer should also check that the associated connection details provide sufficient resistance to the demands associated with the cross-frame or diaphragm loads.

For additional information about the requirement that cross-frames or diaphragms for the routine steel I-girder bridges covered by this Guide also be designed to satisfy the stability bracing strength and stiffness requirement, consult Section 6.6.3.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures and the NSBA's Steel Bridge Design Handbook – Chapter 13: Bracing System Design for further information on these requirements and their application to cross-frames and diaphragms in steel I-girder bridges. In addition, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge includes an example of the design of a truss-type cross-frame for a routine steel I-girder bridge with consideration of these stability bracing strength and stiffness requirements. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.7.4.3 Composite Box-Section Members

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies design requirements for cross-frames and diaphragms in bridges containing composite steel box-section members. The routine steel-girder bridges covered by this Guide are not comprised of composite steel box-section members; therefore, the provisions of this Article are not applicable.

6.7.4.4 Noncomposite Box-Section Members

6.7.4.4.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article contains general design requirements for diaphragms used in noncomposite steel box-section members. The routine steel-girder bridges covered by this Guide do not utilize noncomposite steel box-section members; therefore, the provisions of this Article are not applicable.

6.7.4.4.2 Square and Rectangular HSS Members

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article contains design requirements for diaphragms used in noncomposite steel square and rectangular HSS (Hollow Structural Section) members. The routine steel-girder bridges covered by this Guide do not utilize noncomposite steel square and rectangular HSS members; therefore, the provisions of this Article are not applicable. The use of such members in routine steel I-girder bridges is impractical and uneconomical. There are no provisions or precedent for using these types of sections as the main spanning elements in routine bridges. Furthermore, these types of members are more costly to fabricate, and involve more complicated and expensive connection details than the single-angle members typically used in cross-frames.

6.7.4.4.3 Welded and Nonwelded Built-Up Noncomposite Box-Section Members

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article contains design requirements for diaphragms and cross-frames used in welded and nonwelded built-up noncomposite steel box-section members. The routine steel-girder bridges covered by this Guide do not utilize welded or nonwelded built-up noncomposite steel box-section members as either the main spanning member or as bracing members; therefore, the provisions of this Article are not applicable.

6.7.4.5 Trusses and Arches

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies design requirements for diaphragms used in truss and arch bridges. The routine steel-girder bridges covered by this Guide do not utilize trusses or arches; therefore, the provisions of this Article are not applicable.

6.7.5 Lateral Bracing

6.7.5.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article specifies general design requirements for lateral bracing in steel bridges of various types. Lateral bracing is defined in the AASHTO LRFD BDS as a truss placed in a horizontal plane between two I-girders or two flanges of a tub girder to maintain cross-sectional geometry and to provide additional stiffness and stability to the bridge system. Lateral bracing is typically used to control loads and deflections due to wind in longer-span structures (particularly in the fully erected steelwork prior to the casting of the concrete deck), to transfer superstructure seismic loads to the supports, and to control cross-section geometry and provide global stability during fabrication, erection and deck placement for longer-span straight or curved structures or narrow straight or curved I-girder bridge units with three or fewer girders. Most of the routine steel I-girder bridges, as defined for the purposes of this Guide do not need lateral bracing to reduce horizontal displacements under wind loading or improve stability during construction.

However, in unusual situations the use of a limited amount of top flange lateral bracing near the ends of the span may represent an effective and practical way to improve stability or control horizontal wind deflections. For instance, moderately or extremely narrow single span routine steel I-girder bridges with span lengths in excess of approximately 160 feet may exhibit insufficient stability during construction; the use of top flange lateral bracing is one possible solution (see the Discussion of Article 6.7.4.2.2 about stability bracing requirements for steel I-girder bridges). Similarly, single span bridges with span lengths near the upper end of the 200-foot span length limit of the routine steel I-girder bridges considered in this Guide may exhibit excessive horizontal displacement under wind loading during construction; the use of top flange lateral bracing is one possible solution (see the Discussion of Article 3.4.2.2 about evaluation of deflections at the service limit state).

For further information on lateral bracing, interested readers are encouraged to consult Section 6.3.2.10 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.7.5.2 I-Section Members

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article specifies design requirements for lateral bracing (see the Discussion of Article 6.7.5.1 in this Guide) in bridges containing steel I-section members. Most of the routine steel I-girder bridges, as defined for the purposes of this Guide do not need lateral bracing to reduce horizontal displacements under wind loading or improve stability during construction.

However, in unusual situations the use of a limited amount of top flange lateral bracing near the ends of the span may represent an effective and practical way to improve stability or control horizontal wind deflections. For instance, moderately or extremely narrow single span routine steel I-girder bridges with span lengths in excess of approximately 160 feet may exhibit insufficient stability during construction; the use of top flange lateral bracing is one possible solution (see the Discussion of Article 6.7.4.2.2 about stability bracing requirements for steel I-girder bridges). Similarly, single span bridges with span lengths near the upper end of the 200-foot span length limit of the routine steel I-girder bridges considered in this Guide may exhibit excessive horizontal displacement under wind loading during construction; the use of top flange lateral bracing is one possible solution (see the Discussion of Article 3.4.2.2 about evaluation of deflections at the service limit state).

6.7.5.3 Tub Section Members

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies design requirements for lateral bracing (see the Discussion of Article 6.7.5.1 in this Guide) in bridges containing composite steel tub-section members. The routine steel-girder bridges covered by this Guide are not comprised of composite steel tub-section members; therefore, the provisions of this Article are not applicable.

6.7.5.4 Trusses

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies design requirements for lateral bracing (see the Discussion of Article 6.7.5.1 in this Guide) in truss bridges. The routine steel-girder bridges covered by this Guide do not utilize trusses; therefore, the provisions of this Article are not applicable.

6.7.6 Pins

6.7.6.1 Location

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides a recommendation that pins should be located to minimize the force effects due to eccentricity. Pins are found on older existing bridges and are not used on modern steel girder bridges. For example, older multi-span steel I-girder bridge designs sometimes used pin-and-hanger or similar details to create hinges and impose statically determinate behavior in the superstructure to simplify analysis of the bridge. Advancements in analytical techniques and software have long since made it easier to design a multi-span continuous superstructure, negating the need to force the articulation of the structure to be statically determinate. Pin and hanger details have proven to be problematic details subject to corrosion issues and lack of redundancy. The use of pins in routine steel I-girder bridges offers no benefits and is strongly discouraged. Therefore, the provisions in this Article are not applicable.

6.7.6.2 Resistance

6.7.6.2.1 Combined Flexure and Shear

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article cover the resistance of pins subject to combined flexure and shear. Pins are found on older existing bridges and are not used on modern steel girder bridges. For example, older multi-span steel I-girder bridge designs sometimes used pin-and-hanger or similar details to create hinges and impose statically determinate behavior in the superstructure to simplify analysis of the bridge. Advancements in analytical techniques and software have long since made it easier to design a multi-span continuous superstructure, negating the need to force the articulation of the structure to be statically determinate. Pin and hanger details have proven to be problematic details subject to corrosion issues and lack of redundancy. Pins were also used in older steel rocker bearing designs; these types of bearings have demonstrated adverse maintenance characteristics and poor performance. The use of pins in routine steel I-girder bridges offers no benefits and is strongly discouraged. Therefore, the provisions in this Article are not applicable.

6.7.6.2.2 Bearing

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article covers the computation of the factored bearing resistance on pins. Pins are found on older existing bridges and are not used on modern steel girder bridges. For example, older multi-span steel I-girder bridge designs sometimes used pin-and-hanger or similar details to create hinges and impose statically determinate behavior in the superstructure to simplify analysis of the bridge. Advancements in analytical techniques and software have long since made it easier to design a multi-span continuous superstructure, negating the need to force the articulation of the structure to be statically determinate. Pin and hanger details have proven to be problematic details subject to corrosion issues and lack of redundancy. Pins were also used in older steel rocker bearing designs; these types of bearings have demonstrated adverse maintenance characteristics and poor performance. The use of pins in routine steel I-girder bridges offers no benefits and is strongly discouraged. Therefore, the provisions in this Article are not applicable.

6.7.6.3 Minimize Size Pin for Eyebars

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies the minimum diameter of the pin used with eyebars. Eyebars are found on older existing bridges and are not used on modern steel girder bridges. Pins are found on older existing bridges and are not used on modern steel girder bridges. For example, older multi-span steel I-girder bridge designs sometimes used pin-and-hanger or similar details to create hinges and impose statically determinate behavior in the superstructure to simplify analysis of the bridge. Advancements in analytical techniques and software have long since made it easier to design a multi-span continuous superstructure, negating the need to force the articulation of the structure to be statically determinate. Pin and hanger details have proven to be problematic details subject to corrosion issues and lack of redundancy. Pins were also used in older steel rocker bearing designs; these types of bearings have demonstrated adverse maintenance characteristics and poor performance. The use of pins in routine steel I-girder bridges offers no benefits and is strongly discouraged. Therefore, the provisions in this Article are not applicable.

6.7.6.4 Pins and Pin Nuts

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The Article specifies material requirements for pins and pin nuts and the types of nuts that may be used to secure a full bearing of all parts connected upon the turned body of a pin. Pins are found on older existing bridges and are not used on modern steel girder bridges. For example, older multi-span steel I-girder bridge designs sometimes used pin-and-hanger or similar details to create hinges and impose statically determinate behavior in the superstructure to simplify analysis of the bridge. Advancements in analytical techniques and software have long since made it easier to design a multi-span continuous superstructure, negating the need to force the articulation of the structure to be statically determinate. Pin and hanger details have proven to be problematic details subject to corrosion issues and lack of redundancy. Pins were also used in older steel rocker bearing designs; these types of bearings have demonstrated adverse maintenance characteristics and poor performance. The use of pins in routine steel I-girder bridges offers no benefits and is strongly discouraged. Therefore, the provisions in this Article are not applicable.

6.7.7 Heat-Curved Rolled Beams and Welded Plate Girders

6.7.7.1 Scope

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies the grades of structural steel in the ASTM A709/A709M Specification that may be used for rolled beams and constant-depth welded I-section plate girders that are heat-curved to obtain a horizontal curvature.

The routine steel I-girder bridges covered by this Guide are not horizontally curved; therefore, the provisions in this Article are not applicable.

6.7.7.2 Geometric Limitations

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article refers to Article 14.3.2.2 of the AASHTO *LRFD Steel Bridge Fabrication Specifications* for the cross-sectional and radius limitations on heat-curving of rolled beams and constant-depth welded I-section plate girders to obtain a horizontal curvature. The Engineer is to indicate on the contract documents whether heat curving is permitted according to these limitations.

The routine steel I-girder bridges covered by this Guide are not horizontally curved; therefore, the provisions in this Article are not applicable.

6.7.8 Bent Plates

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

Bent plate diaphragms are sometimes used on routine steel I-girder bridges. The minimum bending radius and direction of final rolling of the plate are important design parameters to be specified in the contract documents. This article directs the Engineer to Section 10 of the *AASHTO LRFD Steel Bridge Fabrication Specifications* for fabrication requirements related to cold or hot bending and provides Commentary with additional information (the requirements for cold bending, which is typically used, specified in Article 10.2 of the *AASHTO LRFD Steel Bridge Fabrication Specifications* are also summarized in the Commentary for this Article in the AASHTO LRFD BDS). The discussion of minimum bend radii is particularly helpful to designers when detailing bent plate diaphragms.

In addition to these specifications, the Engineer should review the AASHTO/AWS D1.5/D1.5M *Bridge Welding Code* requirements and any applicable Owner-agency specifications related to the bending of plates.

6.8 TENSION MEMBERS

6.8.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article is applicable to the design of tension members (i.e., members subject to axial tension) and the design of connection elements subject to tension (see the Discussion of Article 6.13.5.2 in this Guide). For the routine steel I-girder bridges covered by this Guide, these provisions are applicable to the design of cross-frame members subject to axial tension and to the design of flange splice plates and cross-frame gusset plates subject to tension.

The factored tensile resistance of a member or connection element at the strength limit state is to be taken as the lesser of the resistance based on yielding on the gross section or fracture on the net section when there are holes present (see the Discussion of Article 6.8.2.1 in this Guide). This Article lists some important considerations when calculating the factored tensile resistance, which are summarized as follows:

- Only holes larger than standard holes for connectors such as bolts need to be deducted from the gross section. Such holes would include pin holes, access holes, and perforations.
- When calculating the net area, all holes are to be deducted from the section. The correction factor for staggered holes is to be considered when deducting the area of connector holes (see the Discussion of Article 6.8.3 in this Guide).

- The reduction factor, *U*, specified in Article 6.8.2.2 for tension members and Article 6.13.5.2 for connection elements is to be considered to account for the effect of shear lag in the determination of the net section fracture resistance (see the Discussion of Articles 6.8.2.2 and 6.13.5.2 in this Guide).
- The 85-percent maximum area efficiency factor for connection elements specified in Article 6.13.5.2 must be considered to provide reserve capacity to account for limited inelastic deformation capabilities due to the relatively small length of the connection elements (see the Discussion of Article 6.13.5.2 in this Guide).
- Slenderness requirements specified in Article 6.8.4 must be satisfied (see the Discussion of Article 6.8.4 in this Guide).
- Fatigue requirements specified in Article 6.6.1 must be satisfied (see the Discussion of Article 6.6.1 in this Guide).
- Block shear requirements specified in Article 6.13.4 must be satisfied at end connections (see the Discussion of Article 6.13.4 in this Guide).

For further information on the design of tension members, consult Section 6.6.3.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. Section 8.4.1 of NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge includes an example cross-frame design.

In addition to the minimum design requirements specified in Article 6.7.4.1 (see the Discussion of Article 6.7.4.1 in this Guide), cross-frames or diaphragms for the routine steel I-girder bridges covered by this Guide are also be designed to satisfy the stability bracing strength and stiffness requirements specified in Article 6.7.4.2.2 (see the Discussion of Article 6.7.4.2.2 in this Guide). Consult Section 6.6.3.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures, the NSBA's Steel Bridge Design Handbook – Chapter 13: Bracing System Design for further information on these requirements and their application to cross-frames and diaphragms in steel I-girder bridges. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.8.2 Tensile Resistance

6.8.2.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article define the factored tensile resistance, P_r, of a tension member or a connection element subject to tension at the strength limit state as the lesser of the tensile resistance for yielding on the gross section or fracture on the net section (when there are holes present) given by Eqs. 6.8.2.1-1 and 6.8.2.1-2, respectively. For the routine steel I-girder bridges covered by this Guide, these provisions are applicable to the design of cross-frame members subject to axial tension and to the design of flange splice plates and cross-frame gusset plates subject to tension.

The Commentary for this article discusses application of the shear lag reduction factor, U, (see the Discussion of Article 6.8.2.2 in this Guide) and the bolt hole reduction factor, R_p, that accounts for the reduced fracture resistance in the vicinity of bolt holes punched full size. Article 6.6.1.2.3 specifies that unless information is available to the contrary, bolt holes in bracing members and their connection plates are to be assumed for design to be punched full size (see the Discussion of Article 6.6.1.2.3 in this Guide). Bracing member connections are often punched full size, whereas bolt holes in splice connections are typically drilled full size; the Engineer is encouraged to consult with the Fabricator regarding the fabrication of the bolt holes. The factors U and R_p only apply when computing the net section fracture resistance at the strength limit state.

The Commentary also contains a narrative on the rational of using both the tensile resistance for yielding on the gross section and fracture on the net section to bound the expected behavior of the steel to provide both reliable strength and behavior and limit excessive deformations.

For further information on the design of tension members, consult Section 6.6.3.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. Section 8.4.1 of NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge includes an example cross-frame design.

In addition to the minimum design requirements specified in Article 6.7.4.1 (see the Discussion of Article 6.7.4.1 in this Guide), cross-frames or diaphragms for the routine steel I-girder bridges covered by this Guide are also be designed to satisfy the stability bracing strength and stiffness requirements specified in Article 6.7.4.2.2 (see the Discussion of Article 6.7.4.2.2 in this Guide). Consult Section 6.6.3.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures, and the NSBA's Steel Bridge Design Handbook – Chapter 13: Bracing System Design for further information on these requirements and their application to cross-frames and diaphragms in steel I-girder bridges.

The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.8.2.2 Reduction Factor, U

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article deal with the computation of the reduction factor, U, to account for shear lag effects associated with end connection geometry when computing the net section fracture resistance of a tension member or connection element subject to tension at the strength limit state (see the Discussion of Articles 6.8.2.1 and 6.13.5.2 in this Guide).

Shear lag is a consideration when the tensile force in the member or element is applied eccentrically or transmitted to some, but not all, of the connection elements; e.g., an angle having a connection to only one leg or when the connection elements do not lie in a common plane. In such cases, the tensile force is not uniformly distributed over the net area and the critical net section may not be fully effective.

In lieu of a refined analysis, which is not recommended for the design of the routine steel I-girder bridges covered by this Guide, the case figures provided in Table 6.8.2.2-1 may be used to determine the reduction factor, U, to be applied for a given scenario. Example calculations of the terms, $\bar{x}$ and L, are given in Figure C6.8.2.2-1. The connection length, L, shown in the figure is defined for general cases as the maximum average length of the longitudinal welds or the out-to-out distance between the bolts in the connection parallel to the line of force. For members with combinations of longitudinal and transverse welds, L is the maximum average length of the longitudinal welds. The transverse weld does not significantly affect the fracture resistance based on shear lag. In some cases, a single prescribed value of the reduction factor, U, is listed, while in other cases, an equation for the calculation of the value of the reduction factor, U, is provided.

Table 6.8.2.2-1 should be read very carefully when choosing the correct value of the reduction factor, U, as the "Description of Element" can be confusing. Readers are also directed to the related provisions and commentary in AISC's Specifications for Structural Steel Buildings and Commentary (specifically Article D3, Table D3.1, and the Commentary for Article D3, which has helpful discussion and figures). Several scenarios commonly found in routine steel I-girder bridges are listed below, with their corresponding Case number in Table 6.8.2.2-1.

- *Design of bolted flange splice plates:* Case 1 – The connection elements essentially lie in a common plane and are fully connected (see the Discussion of Article 6.13.5.2 in this Guide).

- *Design of cross-frame gusset plates bolted to cross-frame connection plates (stiffeners):* Case 1 - The connection elements essentially lie in a common plane and are fully connected.

- *Design of single angle, double angle, or tee (WT) members attached to a gusset plate using welds (a common and recommended detail for truss-type cross-frames in routine steel I-girder bridges is to use a combination of longitudinal welds along the length of the connection and a transverse weld across the end of the connection – see page 108 of AASHTO-NSBA Steel Bridge Collaboration's Guideline G1.4-2006 Guidelines for Design Details and Figure 2.2.6.1 of AASHTO-NSBA Steel Bridge Collaboration's Guideline G12.1-2020 Guidelines to Design for Constructability and Fabrication): Case 2 – The* connection occurs only through one flange of the angle or only through the flanges of the tee. See Figure C6.8.2.2-1 for the determination of $\bar{x}$ and L. For longitudinal welds with unequal lengths, the average length of the longitudinal welds is to be used for L. The length of the transverse weld is not included in the computation of L.

- *Design of single angle, double angle, or tee (WT) members attached to a gusset plate or to a cross-frame connection plate (stiffener) using a bolted connection:* Case 2, or alternately Case 7 for WT members or Case 8 for angles– See Figure C6.8.2.2-1 for the determination of $\bar{x}$ and L.
- *Design of wide-flange or channel shapes attached to a gusset plate or to a cross-frame connection plate (stiffener) using a bolted connection through their webs:* Case 2, or alternately Case 7 for wide-flange shapes only – See Figure C6.8.2.2-1 for the determination of $\bar{x}$ and L.
- *Design of gusset plates attached to cross-frame connection plates (stiffeners) using only longitudinal welds along the length of the connection (with no transverse weld across the end of the connection):* Case 4 –For longitudinal welds with unequal lengths, the average length of the longitudinal welds is to be used for L; the length of each weld is not to be less than four times the weld size. See Figure C6.8.2.2-1 for the determination of $\bar{x}$ and L.

As noted in the Article, for open-section members, e.g., angles, tees, wide-flange shapes, and channels, the calculated value of *U* from the table is not to be taken less than the ratio of the gross area of the connected element or elements to the member gross area; for the typical cases of single angles, double angles, or tee (WT) sections welded to gusset plates or bolted directly to the cross-frame connection plate (stiffener), the "connected element" is taken as the angle, double angle, or tee member (as applicable) and the "member" is taken as the gusset plate or connection plate (as applicable). It is conservative to neglect this check of the ratio of the areas of the connected element and the member, as they only define a lower bound to the value of the shear reduction factor, *U*; using a lower value, calculated using Table 6.8.2.2-1, would be conservative.

The Commentary for this Article states the moment resulting from the eccentricity between the member and the connection plate need not be considered in the design of the member or connection plates for angle members and light structural tee members loaded eccentrically in axial tension; the effect of the connection eccentricity is addressed through the use of the shear lag reduction factor, *U*.

For further information on the shear lag reduction factor, *U*, consult Section 6.6.3.3.2.4 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. Section 8.4.1 of NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge includes an example cross-frame design.

The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.8.2.3 Combined Axial Tension, Flexure, and Flexural and/or Torsional Shear

6.8.2.3.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article address the strength interaction for any combination of axial tension, uniaxial or biaxial flexure, and flexural and/or torsional shear at the strength limit state, including combinations where one or more of the individual actions may be zero. Eqs. 6.8.2.3.1-1 and 6.8.2.3.1-2 represent the stability and overall strength interaction effects for uniaxial or biaxial bending combined with axial tension and general yielding under axial tension and flexure (with the relationship represented by these equations shown in Figure C6.8.2.3.1-1). The alternative Eqs. 6.8.2.3.1-3 and 6.8.2.3.1-4 conservatively recognize that axial tension tends to have a negligible to beneficial impact on the flexural resistances associated with compression buckling (with the relationship represented by these equations shown in Figure C6.8.2.3.1-2). The Commentary for this Article addresses the overall length effects associated with the lateral-torsional buckling resistance when the flexural resistance about the x-axis is influenced by lateral-torsional buckling and how these effects should be considered in combination with other cross-section based resistance checks within these relationships.

This Article is not applicable to the design of I-sections used as the main spanning elements in a routine steel I-girder bridge. Such members in routine steel I-girder bridges are not tension members. The interaction of flange flexural shear stresses with the axial and flexural resistances of the member is assumed to be negligible in I-section members in the AASHTO LRFD BDS. The interaction between torsional and/or flexural shear stresses in I-section member flanges with other members resistances is also neglected. Moment-shear strength interaction in the presence of low (or zero) levels of axial force is also small and may be neglected in I-section members. The tensile forces occurring in the flanges when such members are subject to major axis bending moments are already addressed in Article 6.10 and its associated sub-Articles.

This Article is also not applicable to the design of truss-type cross-frame members subject to axial tension in routine steel I-girder bridges. The moment resulting from the eccentricity between the member and the connection plate need not be considered in the design of the member or connection plates for angle members and light structural tee (WT) members loaded eccentrically in axial tension; the effect of the connection eccentricity is addressed through the use of the shear lag reduction factor, U (see the Discussion of Article 6.8.2.2 in this Guide).

6.8.2.3.2 Interaction with Torsional and/or Flexural Shear

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article address torsional and/or flexural shear interaction with axial tension at the strength limit state. These provisions are applicable only for noncomposite rectangular box-section members, including square and rectangular HSS (Hollow Structural Sections), and noncomposite circular tubes, including round HSS, subject to fairly significant levels of torsion (i.e., $f_{ve}/\phi_T F_{cv}$ greater than 0.2, where the terms are defined in Article 6.9.2.2.2) in combination with flexure and/or axial tensile force, and/or to I-section members and the preceding members subject to flexural shear in combination with significant levels of axial tensile force (i.e., P_u/P_{ry} greater than 0.05, where the terms are defined in Article 6.8.2.3.1). Such members are not used in the design of the routine steel I-girder bridges covered by this Guide. Cross-frame members in routine steel I-girder bridges are subject to significant axial force, but generally are not subject to any significant torsional or flexural shear effects. Therefore, the provisions of this Article are not applicable to the routine steel I-girder bridges covered by this Guide.

6.8.2.3.3 Tension Rupture Under Axial Tension or Compression Combined with Flexure

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article address the strength interaction between flexure and axial tension or compression pertaining to tension rupture at certain specified locations. These locations include: 1) cross-sections containing bolt holes in one or more flanges subject to tension under combined axial tension or compression and flexure at connection or nonconnection locations; 2) cross-sections at connection or nonconnection locations subject to combined axial tension and flexure and containing bolt holes in other cross-section elements; and 3) cross-sections at welded connections subject to combined axial tension and flexure. The equation in this Article focuses on the specific axial force, tension or compression, combined with the specific moment at the cross section under consideration. Axial compressive forces on the cross-section containing the bolt holes result in a negative force ratio and produce a beneficial subtractive effect, whereas axial tensile forces result in a positive force ratio and produce an additive effect.

In terms of cross-frame members subject to axial tension in the routine steel I-girder bridges covered by this Guide, the moment resulting from the eccentricity between the member and the connection plate need not be considered in the design of the member or connection plates for angle members and light structural tee members loaded eccentrically in axial tension; the effect of the connection eccentricity is addressed through the use of the shear lag reduction factor, U (see the Discussion of Article 6.8.2.2 in this Guide).

These provisions may be applicable to tee-section or double-angle cross-frame members if they are subject to tension under combined axial compression and flexure. It should be noted that using rolled steel tee (WT) and double-angle sections as cross-frame members for routine steel I-girder bridges is generally discouraged, while the use of single-angle sections is encouraged. The magnitude of the forces in cross-frame members in routine steel I-girder bridges covered by this Guide is generally small enough such that single-angle members are adequate. Cross-frame forces are typically only large enough to warrant the use of rolled steel tee (WT) or double-angle sections in curved and/or skewed steel I-girder bridges. Rolled steel tee (WT) sections are typically quite expensive to fabricate. Tee (WT) sections are cut from full wide-flange (W) shapes and generally require straightening after the cutting process, which adds significant fabrication effort and cost. Double-angle sections are often viewed as problematic from a maintenance perspective; the surfaces between the adjacent angle flanges are difficult or impossible to paint in the field, and/or can suffer from potentially severe pack rust.

6.8.3 Net Area

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article defines the net area, A_n, of a member or connected element when holes are present. The net area of each element in a given cross-section is defined as the product of the thickness of the member or connected element and its smallest net width. The overall net area of a given cross-section, A_n, is used in the computation of the net section fracture resistance and the block shear rupture resistance when the member or connection element is subject to tension (see the Discussion of Articles 6.8.2.1, 6.13.4, and 6.13.5.2 in this Guide). The net width of each element in a given cross-section is based on its gross width, reduced to account for the presence of holes in the element. For the routine steel I-girder bridges covered by this Guide, the provisions of this Article are applicable to the design of cross-frame members subject to axial tension and to the design of flange splice plates and cross-frame gusset plates subject to tension.

The width of each standard bolt hole is to be taken as the nominal diameter of the hole. The width of oversize and slotted holes, although not recommended for use in the routine steel I-girder bridges covered by this Guide, is taken equal to the nominal diameter or width of the hole, as applicable. Maximum hole sizes for different bolt diameters are specified in Table 6.13.2.4.2-1 (see the Discussion of Article 6.13.2.4.2 in this Guide). For example, for a 7/8-inch diameter bolt, the maximum hole size of a standard hole equal to 15/16 inches is used in the calculation of A_n.

The net width is to be determined for each chain of holes extending across the connected element along any transverse, diagonal or zigzag line. For each chain, the net width is to be determined by subtracting the sum of all holes in the chain from the total width, and then adding back in the quantity $s^2/4g$ for each space between consecutive holes in the chain when the holes are staggered. The term s is the pitch of any two consecutive holes (i.e., the distance between the center of the two holes along the line of force) and g is the gage of the same two holes (i.e., the distance between the adjacent lines of bolts containing the two holes in the direction perpendicular to the line of force). When holes are staggered in both legs of an angle, the gage for holes in opposite adjacent legs is to be taken as the sum of the gages from the back of the angles less the thickness of the angle. For welded connections, A_n is to be taken as the gross area less any access holes within the connection region.

It is conservative to use the least net width in conjunction with the full tensile force to check the member or connected element. Assuming each bolt transfers an equal share of the load whenever the bolts are arranged symmetrically with respect to the centroidal axis of the member or connected element, a less conservative alternative, particularly with staggered holes, is to check each possible chain with a tensile force obtained by subtracting the force removed by each bolt ahead of that chain from the full tensile force.

For example calculations of the net area of a tension member, consult Sections 6.6.3.3.2.3 and 6.6.4.2.5.6.1 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.8.4 Limiting Slenderness Ratio for Tension Members

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article specify limiting values for evaluating the slenderness ratio, or ratio of the unbraced length over the radius of gyration, of primary and secondary members subject to axial tension, or for evaluating the tension slenderness of primary and secondary compression members subject to stress reversal.

The provisions are not applicable to the tension flanges of flexural members such as the I-shaped sections typically used as the main girders in routine steel I-girder bridges. However, these provisions are directly applicable to the proportioning of cross-frame members in routine steel I-girder bridges.

Determination of whether the cross-frame members should be treated as "Primary" or "Secondary" members should be based on a careful review of the definitions provided in Article 6.2, where it states that a "Primary Member" is "A steel member or component that transmits gravity loads through a necessary as-designed load path. These members are therefore subjected to more stringent fabrication and testing requirements; considered synonymous with the term 'main member.'" In routine steel I-girder bridges, the cross-frame members do not meet this definition. Recall that the definition of a routine steel I-girder bridge is limited to straight bridges with little or no support skew. In such bridges, restoring forces are created in the cross-frames due to any differential deflections between the girders but these restoring forces will typically be small since the differential deflections between the girders in routine steel I-girder bridges are small. Thus, the girders will resist the noncomposite dead loads equally in these bridges, neglecting any effect of the deflections resulting from elastic shortening of the cross-frames, which are generally negligible. The distribution of composite dead loads and live loads in these types of bridges occurs primarily through the deck. As a result, the cross-frame members in a routine steel I-girder bridge can be categorized as "Secondary Members" when addressing the provisions of this Article.

As an aside, the cross-frame members in curved and/or significantly skewed steel I-girder bridges do in fact function to transmit gravity loads through a necessary as-designed load path due to the significantly larger differential deflections between the girders in these bridges, and as a result the cross-frames in those types of bridges are categorized as "Primary Members" when addressing the provisions of this Article. While cross-frame members in straight I-girder bridges with significant support skew brace the girders and serve as an additional transverse load path in the bridge system, the internal cross-frame forces in these bridges are not required for equilibrium of the girders; hence, the cross-frames in these bridges should be categorized as "Secondary Members."

These provisions are applicable for evaluating the tension slenderness ratio of cross-frame members in the routine steel I-girder bridges covered by this Guide; only the limiting value of the ratio for secondary members applies.

6.8.5 Built-Up Members

6.8.5.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article contain general requirements for tension members built-up from rolled or welded shapes connected by continuous plates with or without perforations or by tie plates with or without lacing. Provisions are specified in this Article for the welded or bolted connections between the plates and shapes.

Built-up tension members are not typically used in the routine steel I-girder bridges covered by this Guide, except possibly the application of double angle members in cross-frames; the provisions of this Article are applicable in that case, but the use of double-angle sections is strongly discouraged (as discussed below). It should be noted that using rolled steel tee (WT) and double-angle sections as cross-frame members for routine steel I-girder bridges is generally discouraged, while the use of single-angle sections is encouraged. The magnitude of the forces in cross-frame members in routine steel I-girder bridges covered by this Guide is generally small enough such that single-angle members are adequate. Cross-frame forces are typically only large enough to warrant the use of rolled steel tee (WT) or double-angle sections in curved and/or skewed steel I-girder bridges. Rolled steel tee (WT) sections are typically quite expensive to fabricate. Tee (WT) sections are cut from full wide-flange (W) shapes and generally require straightening after the cutting process, which adds significant fabrication effort and cost. Double- angle sections are considered problematic from a maintenance perspective; the surfaces between the adjacent angle flanges are difficult or impossible to paint in the field, and/or can suffer from potentially severe pack rust.

For further information on built-up tension members, consult Section 6.6.3.3.4 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge includes an example cross-frame design. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.8.5.2 Perforated Plates

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article contains provisions specific to the design of perforated plates used to connect rolled or welded shapes in built-up tension members (see the Discussion of Article 6.8.5.1 in this Guide). The routine steel I-girder bridges covered by this Guide do not utilize built-up tension members with perforated plates; therefore, the provisions of this Article are not applicable.

6.8.6 Eyebars

6.8.6.1 Factored Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article are used to compute the factored tensile resistance of the body of an eyebar at the strength limit state. The resistance is based on yielding on the gross section; fracture on the net section does not control because the net section in the head is at least 1.35 times greater than the section in the body. The routine steel I-girder bridges covered by this Guide do not contain eyebars; therefore, the provisions of this Article are not applicable.

6.8.6.2 Proportions

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article specify the proportioning requirements for the head and body of an eyebar and the location and dimensioning requirements of the pin hole in the eyebar. The routine steel I-girder bridges covered by this Guide do not contain eyebars; therefore, the provisions of this Article are not applicable.

6.8.6.3 Packing

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article specify the detailing requirements for an eyebar assembly to prevent corrosion-causing elements from entering the joint, lateral movement on the pin and lateral distortion of the eyebar due to skew, and repeated eyebar contact due to vibration perpendicular to the eyebar plane. The routine steel I-girder bridges covered by this Guide do not contain eyebars; therefore, the provisions of this Article are not applicable.

6.8.7 Pin-Connected Plates

6.8.7.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article apply to the design of pin-connected plates, which should be avoided wherever possible. The factored tensile resistance of such plates at the strength limit state must satisfy the provisions of Article 6.8.2.1 (see the Discussion of Article 6.8.2.1 in this Guide). The routine steel I-girder bridges covered by this Guide do not contain pin-connected plates; therefore, the provisions of this Article are not applicable.

6.8.7.2 Pin Plates

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article specify the factored resistance for bearing on pin-connected plates, P_r, at the strength limit state and also provisions related to the strengthening of the main plate in the region of the hole and attachment of the pin plates to the main plate. The routine steel I-girder bridges covered by this Guide do not contain pin-connected plates; therefore, the provisions of this Article are not applicable.

6.8.7.3 Proportions

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article specify the proportioning requirements for the main plate and pin plates in a pin-connected assembly and the location and dimensioning requirements of the pin hole in the plates. The routine steel I-girder bridges covered by this Guide do not contain pin-connected plates; therefore, the provisions of this Article are not applicable.

6.8.7.4 Packing

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article specify the detailing requirements for a pin-connected assembly to prevent corrosion-causing elements from entering the joints and lateral movement on the pin and lateral distortion of the assembly due to skew. The routine steel I-girder bridges covered by this Guide do not contain pin-connected plates; therefore, the provisions of this Article are not applicable.

6.9 COMPRESSION MEMBERS

6.9.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article is applicable to the design of compression members (i.e., members subject to axial compression). For the routine steel I-girder bridges covered by this Guide, these provisions are applicable to the design of cross-frame members subject to axial compression.

Neither I-girders nor their flanges in routine steel I-girder bridges are treated as compression members or beam columns per se. The combined effects of axial compression and flange lateral bending in the compression flanges of girders and beams are directly addressed in Article 6.10.

Furthermore, the term "composite steel members" in this Article is not meant to imply that the provisions of this Article apply to composite girders and beams. Instead, this term is referring to concrete-filled tubes or pipes or concrete encased steel members subject to axial compression or to combined axial compression and flexure (see the Discussion of Articles 6.9.5 and 6.9.6 in this Guide).

However, cross-frames in routine steel I-girder bridges often feature members subject to axial compression or combined axial compression and flexure. As such, the provisions of Articles 6.9.1, 6.9.2, 6.9.3, and 6.9.4 are directly applicable to their design.

The language in the Commentary of this Article is short, but significant, and should be read carefully. Examples of "significant additional eccentricity" include items such as the offset from the centroid of a cross-frame member to the faying surface between its connection to a cross-frame connection plate; an eccentricity such as this induces moment that should be considered in the design of that member. However, as clearly stated in the Commentary, "imperfections and eccentricities permissible in normal fabrication and erection" are already accounted for in conventional steel column design formulas and do not need to be treated as sources of additional eccentricity-induced moments.

For further information on the design of compression members, consult Section 6.6.3.4 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge includes an example cross-frame design. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.9.2 Compressive Resistance

6.9.2.1 Axial Compression

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

Neither I-girders nor their flanges in routine steel I-girder bridges are treated as compression members or beam columns per se. The combined effects of axial compression and flange lateral bending in the compression flanges of girders and beams are directly addressed in Article 6.10, and the provisions of Article 6.9.2.1 are therefore not applicable to the design of I-sections in the routine steel I-girder bridges covered by this Guide. However, cross-frames in routine steel I-girder bridges often feature members subject to axial compression or combined axial compression and flexure. As such, the provisions of this Article are directly applicable to their design.

This Article specifically outlines the basic load versus resistance equality that must be satisfied. The definition of the nominal compressive resistance, P_n, refers to Articles 6.9.4 and 6.9.5, but only Article 6.9.4 is applicable. Article 6.9.4 addresses the design of noncomposite members, such as those used in typical cross-frames in routine steel I-girder bridges. Article 6.9.5 addresses composite members; as explained in the Discussion of Article 6.9.1 in this Guide, in this context the term "composite members" refers to concrete-filled tube or pipes or concrete encased steel members subject to axial compression or combined axial compression and flexure (see the Discussion of Articles 6.9.5 and 6.9.6 in this Guide).

6.9.2.2 Combined Axial Compression, Flexure, and Flexural and/or Torsional Shear

6.9.2.2.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article address the strength interaction for any combination of axial compression, uniaxial or biaxial flexure, and flexural and/or torsional shear at the strength limit state, including combinations where one or more of the individual actions may be zero. This Article specifically presents the beam-column interaction equations which must be satisfied for certain types of members subject to combined axial compression and flexure.

In terms of cross-frame members subject to axial compression in the routine steel I-girder bridges covered by this Guide, consideration of the moment resulting from the eccentricity between the member and the connection plate is dependent on the shape of the member's cross section.

- Rolled steel single-angle members subject to combined axial compression and flexure about one or both principal axes and meeting a short list of simple connection and loading criteria may be designed as axially loaded compression members for flexural buckling only, as outlined in Article 6.9.4.4 (see the Discussion of Article 6.9.4.4 in this Guide). As a result, the provisions of Article 6.9.2.2 and the associated sub-articles (including this Article, 6.9.2.2.1) are not applicable for the design of these members.

- Rolled steel tee (WT) and double-angle members, on the other hand, subject to combined axial compression and flexure must be designed in accordance with the provisions of this Article.

Channel sections are sometimes used as a top chord member in end cross-frames used to support the edge of the deck at an expansion joint. Such members are typically connected to the deck using shear connectors. The members in end cross-frames in routine steel I-girder bridges are typically not subject to significant axial loads associated with their function as stability bracing for the girders. The main sources of loading in end cross-frames in routine steel I-girder bridges are wind (wind load on superstructure being transferred to the bearings through pier and end cross-frames) and gravity loads from the deck (primarily the wheel loads applied at the edge of deck plus a small amount of deck dead load). The wind loads are transferred to the bearings through the end cross-frame diagonals, which are typically single-angle, double-angle, or tee (WT) members. Wheel loads and dead load from the deck typically produce bending in the top chords and axial compression in the diagonals.

Furthermore, neither I-girders nor their flanges in routine steel I-girder bridges are treated as compression members or beam columns per se. The combined effects of axial compression and flange lateral bending in the compression flanges of girders and beams are directly addressed in Article 6.10.

Consequently, the remainder of this Discussion is exclusively focused on the application of the provisions of this Article to the design of rolled steel tee (WT) and double-angle members subject to axial compression and uniaxial or biaxial flexure.

It should be noted that using rolled steel tee (WT) and double-angle sections as cross-frame members for routine steel I-girder bridges is generally discouraged, while the use of single-angle sections is encouraged. The magnitude of the forces in cross-frame members in routine steel I-girder bridges covered by this Guide is generally small enough such that single-angle members are adequate. Cross-frame forces are typically only large enough to warrant the use of rolled steel tee (WT) or double-angle sections in curved and/or skewed steel I-girder bridges. Rolled steel tee (WT) sections are typically quite expensive to fabricate. Tee (WT) sections are cut from full wide-flange (W) shapes and generally require straightening after the cutting process, which adds significant fabrication effort and cost. Double-angle sections are often viewed as problematic from a maintenance perspective; the surfaces between the adjacent angle flanges are difficult or impossible to paint in the field, and/or can suffer from potentially severe pack rust.

Eqs. 6.9.2.2.1-1 and 6.9.2.2.1-2 represent the stability and overall strength interaction effects for uniaxial or biaxial bending combined with axial compression and general yielding under axial compression and flexure. As explained in the Commentary, these equations provide an "accurate to conservative approximation of the resistances under combined loading for members in which all the cross-section elements are compact" for flexure. The Commentary further explains that Eqs. 6.9.2.2.1-1 and 6.9.2.2-2 are applicable to cases where the axial and flexural stresses in the flange of the tee or the flange legs of the double angles are in compression, such as when the connection is made through the flange of the tee or the flange legs of the angles, which is the typical case when tee and double-angle sections are used as cross-frame members in routine steel I-girder bridges.

Eq. 6.9.2.2.1-3 is only applicable in cases where the toe of the stem or leg of the double-angle section is in flexural compression; in this case the stem or leg is not considered a compact element. It should be emphasized that this equation is only applicable when the toe of the stem or leg is in compression when subject to bending moment only. If the toe of the stem or leg is in flexural tension (i.e., in tension when subject to bending moment only), even if the toe is in compression when the combined effects of axial compression and flexure are considered, then Eqs. 6.9.2.2.1-1 and 6.9.2.2.1-2 apply. Such a condition will rarely, if ever, occur in the cross-frame of a steel I-girder bridge since in virtually all cases the connection is made through the flanges.

These interaction equations are very straightforward in and of themselves and are easy to implement in hand calculations or design spreadsheets. The Commentary explains that the equations presented in this Article are conservative simplifications of more exact, nonlinear equations. These exact, nonlinear equations are rarely, if ever, used in practice. Using them for the design of cross-frames in routine steel I-girder bridges is neither warranted nor recommended; the simplified equations presented in the Article are more than sufficient for cross-frame design purposes.

The Article goes on to require that the factored design moments, M_{ux} and M_{uy}, be determined in a manner which accounts for the second-order magnification of moments due to geometric nonlinear behavior. The Article allows that the effects of second-order moment magnification can be determined either by means of a second-order elastic analysis (i.e., an iterative geometric nonlinear analysis accounting for P-δ effects) or by the approximate single-step method specified in Article 4.5.3.2.2b or a comparable amplification factor-based procedure. For the design of cross-frame members in routine steel I-girder bridges, the use of the refined second-order analysis approach is not justified. The approximate single-step method specified in Article 4.5.3.2.2b is more than sufficient and should be used (see the Discussion of Article 4.5.3.3.2b in this Guide).

Eq. 6.9.2.2.1-5 addresses the case where the nominal flexural resistance about the major axis of the section is determined according to the provisions of Appendix A6. Appendix A6 addresses optional provisions for determining the flexural capacity of composite and noncomposite I-shaped members; Appendix A6 is intended for use in designing the main girders in a routine steel I-girder bridge, not the design of cross-frame members in routine steel I-girder bridges, so this equation is not applicable (see the Discussion of Appendix A6 in this Guide).

The Article further requires that interaction with torsional and/or flexural shear be considered and directs the reader to Article 6.9.2.2.2. This language applies to the design of noncomposite box-section members, which can be subject to significant torsional and/or flexural shear effects in their typical applications (e.g., as arch rib members, straddle bent caps, and so forth). Cross-frame members in routine steel I-girder bridges are not subject to any significant torsional or flexural shear effects; as such this directive is not applicable, and as can be seen elsewhere in this Guide, Article 6.9.2.2.2 in its entirety is considered not applicable to the design of routine steel I-girder bridges.

6.9.2.2.2 Interaction with Torsional and/or Flexural Shear

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article is intended to apply exclusively to the design of noncomposite rectangular box-section members, including square and rectangular hollow structural sections (HSS), and noncomposite circular tubes, including round HSS, or I-section members subject to significant axial force combined with torsional and/or flexural shear. The levels of torsional and/or flexural shear above which their effects much be considered are defined in this Article.

The interaction of flange flexural shear stresses with the axial and flexural resistances of the member is assumed to be negligible in I-section members in the AASHTO LRFD BDS. The interaction between torsional and/or flexural shear stresses in I-section member flanges with other members resistances is also neglected. Moment-shear strength interaction in the presence of low (or zero) levels of axial force is also small and may be neglected in I-section members.

Cross-frame members in routine steel I-girder bridges are subject to significant axial force, but generally are not subject to any significant torsional or flexural shear effects.

As a result, the provisions of this article are not applicable to the design of the routine steel I-girder bridges covered by this Guide.

6.9.3 Limiting Slenderness Ratio for Compression Members

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The proportioning limits specified in this article are intended to limit the slenderness of members subject to axial compression only, or to evaluate the compression slenderness of tension members subject to stress reversal, so that these members can exhibit a reasonable minimum resistance to buckling.

The provisions are not applicable to the compression flanges of flexural members such as the I-shaped sections typically used as the main girders in routine steel I-girder bridges. However, these provisions are directly applicable to the proportioning of cross-frame members in routine steel I-girder bridges; only the limiting value of the ratio for secondary members applies.

Determination of whether the cross-frame members should be treated as "Primary" or "Secondary" members should be based on a careful review of the definitions provided in Article 6.2, where it states that a "Primary Member" is "A steel member or component that transmits gravity loads through a necessary as-designed load path. These members are therefore subjected to more stringent fabrication and testing requirements; considered synonymous with the term 'main member.'" In routine steel I-girder bridges, the cross-frame members do not meet this definition. Recall that the definition of a routine steel I-girder bridge is limited to straight bridges with little or no support skew. In such bridges, restoring forces are created in the cross-frames due to any differential deflections between the girders but these restoring forces will typically be small since the differential deflections between the girders are small. Thus, the girders will resist the noncomposite dead loads equally in these bridges, neglecting any effect of the deflections resulting from elastic shortening of the cross-frames which are generally negligible. The distribution of composite dead loads and live loads in these types of bridges occurs primarily through the deck. As a result, the cross-frame members in a routine steel I-girder bridge can be categorized as "Secondary Members" when addressing the provisions of this Article.

As an aside, the cross-frame members in curved and/or significantly skewed steel I-girder bridges do in fact function to transmit gravity loads through a necessary as-designed load path due to the significantly larger differential deflections between the girders in these bridges, and as a result the cross-frames in those types of bridges are categorized as "Primary Members" when addressing the provisions of this Article. While cross-frame members in straight I-girder bridges with significant support skew brace the girders and serve as an additional transverse load path in the bridge system, the internal cross-frame forces in these bridges are not required for equilibrium of the girders; hence, the cross-frames in these bridges should be categorized as "Secondary Members."

The Article directs the reader to Article 4.6.2.5 for specification of appropriate effective length factors, K. The choice of effective length factor is dependent on the type of section and its end connections. For the single-angle members commonly used as cross-frame members in routine steel I-girder bridges, regardless of end connection, the effective length factor, K, is specified as

1.0 (see the Discussion of Article 6.9.4.4 in this Guide for further information on the checking of the slenderness ratio limits specified in this Article for single-angle members). For other sections, such as rolled steel tee (WT) or double-angle sections which are sometimes used as cross-frame members in routine steel I-girder bridges, if bolted or welded end connections are provided, the effective length factor, K, is specified as 0.750. The case of "pinned connections" in Article 4.6.2.5 is not applicable for the types of members typically used in routine steel I-girder bridges.

The Article also offers an optional approach for determining the radius of gyration using a "notional section." This provision permits neglecting part of the cross-section of the compression member to allow calculation of a more favorable (larger) radius of gyration, solely for the purposes of satisfying the requirements of this Article. The use of this provision is typically neither warranted nor recommended for the design of compression members in truss-type cross-frames for routine steel I-girder bridges. Given the geometric constraints associated with the definition of a routine steel I-girder bridge for the purposes of this Guide, it is highly unlikely that a prudent, more economical design would result from using this provision. Instead, the cross-frame members should be adequately sized to meet the basic slenderness requirements, which are not onerous for typical cross-frame members in routine steel I-girder bridges.

6.9.4 Noncomposite Members

6.9.4.1 Nominal Compressive Resistance

6.9.4.1.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

Neither I-girders nor their flanges in routine steel I-girder bridges are treated as compression members or beam columns per se. The combined effects of axial compression and flange lateral bending in the compression flanges of girders and beams are directly addressed in Article 6.10, and the provisions of this Article are therefore not applicable to the design of I-sections in the routine steel I-girder bridges covered by this Guide. However, cross-frames in routine steel I-girder bridges often feature members subject to axial load. As such, the provisions of this Article are directly applicable to their design.

The provisions of this Article are comprehensive in terms of addressing the nominal compressive resistance of axially loaded members; they address a variety of buckling modes and a variety of cross-sections. The reader is cautioned to read the provisions carefully before using them.

The article specifies that three buckling modes be considered, and that the nominal compressive resistance, P_n, is to be taken as the smallest value. Not all three modes are applicable for all cross-sectional shapes. The three modes include:

- Flexural buckling
- Torsional buckling
- Flexural-torsional buckling

Three categories of cross-sectional shapes are considered. Some of these are not used as axially loaded members in the design of routine steel I-girder bridges. The three categories of cross-sectional shapes include:

- Doubly symmetric members: These are members which are symmetric about both of their primary orthogonal axes. Examples of doubly symmetric members include rolled wide-flange (W shapes) and other I-shaped members (H and S shapes and some plate girders) with equal-size flanges and also closed sections such as round and hollow structural section (HSS) steel tubes. As previously mentioned, I-shaped sections used as main girders in routine steel I-girder bridges are not treated as compression members. Doubly symmetric shapes are not used as cross-frame members in routine steel I-girder bridges. As a result, the provisions in this Article related to doubly symmetric members are not applicable to the design of the routine steel I-girder bridges covered by this Guide.

- Singly symmetric members: These are members which are symmetric about only one of their primary orthogonal axes. Examples of singly symmetric members include I-shaped steel plate girders with unequal-sized flanges, rolled tee (WT) sections, and double-angle sections. As previously mentioned, I-shaped sections used as main girders in routine steel I-girder bridges are not treated as compression members. However, rolled steel tee (WT) sections are sometimes used as cross-frame members in routine steel I-girder bridges, and the provisions of this article are applicable to their design. The use of double-angle sections is discouraged in new routine steel I-girder bridge designs but may be found on older designs. Double-angle sections are considered problematic from a maintenance perspective; the surfaces between the adjacent angle flanges are difficult or impossible to paint in the field, and/or can suffer from potentially severe pack rust.

- Unsymmetric members: These are members which have no axis of symmetry. Examples of unsymmetric members include single-angle sections. Single-angle sections are often used as cross-frame members in routine steel I-girder bridges, and the provisions of this article are applicable to their design.

The equations for nominal resistance, P_n, are grouped for applicability first based on whether the compression member is comprised of nonslender or slender elements. The provisions for determining the slenderness classification are found in Article 6.9.4.2.1 (see the Discussion of Article 6.9.4.2.1 in this Guide). For compression members composed only of nonslender, longitudinally unstiffened elements, the nominal resistance, P_n, is determined using either Eq. 6.9.4.1.1-1 or 6.9.4.1.1-2, depending on the ratio of the nominal yield resistance, P_o, to the elastic critical buckling resistance, P_e. For compression members with cross-sections containing one or more slender elements, the nominal resistance, P_n, is determined in accordance with the provisions of Article 6.9.4.2.2 (see the Discussion of Article 6.9.4.2.2 in this Guide).

Table 6.9.4.1.1-1 provides a handy guide for determining the nature of a given cross-sectional shape, its potential column buckling modes, and the applicable equations, and associated Article and Commentary, for determining the elastic critical buckling resistance, P_e.

Mention of "longitudinally stiffened elements" is not applicable to the design of the cross-frame members used in routine steel I-girder bridges. Members with longitudinally stiffened elements, in the context of this Article, are generally limited to noncomposite box section members, which are not used in routine steel I-girder bridges.

It should be noted that using rolled steel tee (WT) and double-angle sections as cross-frame members for routine steel I-girder bridges is generally discouraged, while the use of single-angle sections is encouraged. The magnitude of the forces in cross-frame members in routine steel I-girder bridges covered by this Guide is generally small enough such that single-angle members are adequate. Cross-frame forces are typically only large enough to warrant the use of rolled steel tee (WT) or double-angle sections in curved and/or skewed steel I-girder bridges. Rolled steel tee (WT) sections are typically quite expensive to fabricate. Tee (WT) sections are cut from full wide-flange (W) shapes and generally require straightening after the cutting process, which adds significant fabrication effort and cost. Double-angle sections are often viewed as problematic from a maintenance perspective; the surfaces between the adjacent angle flanges are difficult or impossible to paint in the field, and/or can suffer from potentially severe pack rust.

For further information about column buckling theory and the nominal compressive resistance of compression members, consult Section 6.6.3.4.2.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge includes an example cross-frame design. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.9.4.1.2 Elastic Flexural Buckling Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article presents the resistance equation for elastic flexural buckling of concentrically loaded compression members. This is the classic Euler buckling equation and can be applied to a number of members in steel structures.

Neither I-girders nor their flanges in routine steel I-girder bridges are treated as compression members or beam columns per se. The combined effects of axial compression and flange lateral bending in the compression flanges of girders and beams are directly addressed in Article 6.10, so the provisions of this Article, 6.9.4.1.2, are not applicable to the design of I-sections in the routine steel I-girder bridges covered by this Guide.

Instead, in routine steel I-girder bridges, elastic flexural buckling is considered primarily in the design of cross-frame members. As explained in Article 6.9.4.1.1, its related Commentary, and its related Discussion in this Guide, flexural buckling is an applicable buckling mode for the sections typically used in cross-frames for routine steel I-girder bridges, which include single-angle sections, tee (WT) sections, and double-angle sections.

As noted in the Commentary for this Article, Eq. 6.9.4.1.2-1 should be used to calculate the critical flexural buckling resistance about both the *x*- and *y*-axes, and the smaller value should be taken as P_e for use in Eq. 6.9.4.1.1-1 or 6.9.4.1.1-2, as applicable. For single-angle members, the effective slenderness ratio, $(K\ell/r)_{eff}$, determined according to the provisions of Article 6.9.4.4, is used in determining the elastic flexural buckling resistance (see the Discussion of Article 6.9.4.4 in this Guide).

For further information on the elastic flexural buckling resistance, consult Section 6.6.3.4.2.3.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge includes an example cross-frame design. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.9.4.1.3 Elastic Torsional Buckling and Flexural-Torsional Buckling Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article presents resistance equations for elastic torsional buckling and flexural-torsional buckling of concentrically loaded compression members.

Eq. 6.9.4.1.3-1 is the resistance equation for torsional buckling of open-section doubly symmetric members, such as doubly symmetric I-shaped members. Neither I-girders nor their flanges in routine steel I-girder bridges are treated as compression members or beam columns per se. The combined effects of axial compression and flange lateral bending in the compression flanges of girders and beams are directly addressed in Article 6.10, so Eq. 6.9.4.1.3-1 is generally not applicable to the design of the routine steel I-girder bridges covered by this Guide.

Eqs. 6.9.4.1.3-2 through 6.9.4.1.3-6 address calculation of the flexural-torsional buckling resistance of open-section singly symmetric members, such as rolled tee (WT), double-angle, and channel members. These members are sometimes used in cross-frames or as diaphragms for routine steel I-girder bridges and these equations must be checked for these members in such cases when subject to axial compression.

Eqs. 6.9.4.1.3-7 through 6.9.4.1.3-9 address calculation of the flexural-torsional buckling resistance of open-section unsymmetric members, such as single-angle members. These members are often used in cross-frames for routine steel I-girder bridges. However, when the effective slenderness ratio, $(K\ell/r)_{eff}$, determined according to the provisions of Article 6.9.4.4, is used in place of $(K\ell/r_s)$ in determining the nominal flexural resistance, P_n, of single-angle members, which should always be done for single-angle members loaded in combined axial compression and flexure, flexural-torsional buckling need not be checked (see the Discussion of Article 6.9.4.4 in this Guide).

The equations are lengthy but are relatively straightforward and are a function of simple material and geometric parameters. Many of the geometric parameters are section property variables; the section properties for common rolled steel tee (WT), double-angle, and channel sections can be found in AISC's Database of Rolled Steel Shape Section Properties. Designers can easily program the resistance equations and integrate the database into a spreadsheet to automate the calculation of the flexural-torsional buckling resistance of tee (WT), double-angle, or channel sections.

The Commentary for this Article provides a thorough explanation of flexural-torsional buckling, including helpful tips for simplifying the calculations and for determining when certain buckling modes may not control.

It should be noted that using rolled steel tee (WT) and double-angle sections as cross-frame members for routine steel I-girder bridges is generally discouraged, while the use of single-angle sections is encouraged. The magnitude of the forces in cross-frame members in routine steel I-girder bridges covered by this Guide is generally small enough such that single-angle members are adequate. Cross-frame forces are typically only large enough to warrant the use of rolled steel tee (WT) or double-angle sections in curved and/or skewed steel I-girder bridges. Rolled steel tee (WT) sections are typically quite expensive to fabricate. Tee (WT) sections are cut from full wide-flange (W) shapes and generally require straightening after the cutting process, which adds significant fabrication effort and cost. Double-angle sections are often viewed as problematic from a maintenance perspective; the surfaces between the adjacent angle flanges are difficult or impossible to paint in the field, and/or can suffer from potentially severe pack rust.

For further information about the elastic torsional buckling and flexural-torsional buckling resistance, and for more information about tee (WT), double-angle, and channel sections, consult Sections 6.6.3.4.2.3.4, 6.6.3.5.3, and 6.6.3.5.4, respectively, of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. Note that this Manual is based on AASHTO LRFD BDS 7th Edition (2014, with Interim Revisions through 2015) provisions for double-angle and tee (WT) sections, but the explanations of general concepts are still valid. NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge includes an example cross-frame design. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.9.4.2 Effects of Local Buckling on the Nominal Compressive Resistance

6.9.4.2.1 Classification of Cross-Section Elements

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article presents the requirements for determining whether longitudinally unstiffened cross-section elements of members subject to axial compression can be considered nonslender. The provisions are applicable to a wide variety of cross-sectional elements in both open and closed sections. For the routine steel I-girder bridges covered by this Guide, the provisions of this Article are applicable to those members which are commonly used in their design.

The Commentary for this Article explains that, "Compression members with cross-sections composed only of nonslender longitudinally unstiffened elements are able to develop their full yield strength under uniform axial compression without any significant impact from local buckling." In other words, local buckling will not adversely affect the overall compressive resistance of those types of sections. Table 6.9.4.2.1-1 provides a handy summary of the applicable width-to-thickness or slenderness ratio limits defining a nonslender element for a wide variety of longitudinally unstiffened cross-sectional elements. The element widths, *b*, to be applied in checking these limits are also provided in the table.

The width-to-thickness and slenderness ratio limits are based on simple equations which are easily calculated by hand or programmed in spreadsheets. Many AISC rolled shapes are proportioned such that their cross-section elements meet these slenderness limits, and they can be classified as nonslender elements, but designers should check the cross-sectional elements of any member they are designing as a standard practice.

For further information on slender and nonslender member elements, consult Section 6.6.3.4.2.4 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge includes an example cross-frame design. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.9.4.2.2 Slender Longitudinally Unstiffened Cross-Section Elements

6.9.4.2.2a General

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article presents the equation for the nominal compressive resistance, P_n, of compression members whose cross-sections are composed of one or more longitudinally unstiffened slender elements. The classification of a cross-sectional element as slender or nonslender is covered in Article 6.9.4.2.1 (see the Discussion of Article 6.9.4.2.1 in this Guide).

Neither I-girders nor their flanges in routine steel I-girder bridges are treated as compression members or beam columns per se. The combined effects of axial compression and flange lateral bending in the compression flanges of girders and beams are directly addressed in Article 6.10, so the provisions of this Article are not applicable to the design of I-sections used as main spanning elements in the routine steel I-girder bridges covered by this Guide.

Cross-frame members may have a cross-section composed of one or more slender longitudinally unstiffened elements, in which case the provisions of this Article would be applicable. The stems of a significant number of rolled tee sections and one or both legs of many rolled angle sections are typically classified as slender elements. In such cases, the provisions of Article 6.9.4.2.2b may also be applicable (see the Discussion of Article 6.9.4.2.2b in this Guide).

Eqs. 6.9.4.2.2a-1 and 6.9.4.2.2a-2 define the nominal compressive resistance, P_n, as a function of the critical (or smallest) buckling resistance based on flexural, torsional, or flexural-torsional buckling of the overall member, reduced to account for the adverse impacts of local buckling of any longitudinally unstiffened slender elements in the cross-section. The reduction factor is essentially the ratio of the effective area to the gross area (A_{eff} /A_g) of the cross-section. The effective area of the cross-section elements, A_{eff}, generally reflects reductions in the effective width of any slender elements in the cross-section, as defined in Article 6.9.4.2.2b for all sections except circular tubes and round Hollow Structural Shapes (HSS); the effective area, A_{eff}, for those types of shapes is addressed in Article 6.9.4.2.2c. Where necessary, Eq. 6.9.4.2.2a-3 should be used to calculate A_{eff} for the rolled sections used as cross-frame members in the routine steel I-girder bridges covered by this Guide.

For further information on slender member elements, consult Section 6.6.3.4.2.4.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. Note that this Manual is based on AASHTO LRFD BDS 7th Edition (2014, with Interim Revisions through 2015) provisions for slender and nonslender elements, but the explanations of general concepts are still valid. NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge includes an example cross-frame design. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.9.4.2.2b Effective Width of Slender Elements

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article defines the effective width of slender longitudinally unstiffened elements in members subject to axial compression. See the Discussion of Article 6.9.4.2.2a in this Guide for further information on slender longitudinally unstiffened elements. A wide variety of elements are addressed, many of which are not (and should not be) used in the routine steel I-girder bridges covered by this Guide.

Neither I-girders nor their flanges in routine steel I-girder bridges are treated as compression members or beam columns per se. The combined effects of axial compression and flange lateral bending in the compression flanges of girders and beams are directly addressed in Article 6.10, so the provisions of this Article are not applicable to the design of I-sections used as main spanning elements in the routine steel I-girder bridges covered by this Guide.

Cross-frame members may have a cross-section composed of one or more slender longitudinally unstiffened elements, in which case the provisions of this Article would be applicable. The stems of a significant number of rolled tee sections and one or both legs of many rolled angle sections are typically classified as slender elements. In such cases, the provisions of this Article are applicable. The effective width, b_e, of a given slender element is used in Article 6.9.4.2.2a to calculate the effective area, A_{eff}, used in the calculation of the nominal compressive resistance, P_n, of the overall member, accounting for the adverse effects of local buckling of any slender elements in the cross-section (see the Discussion of Article 6.9.4.2.2a in this Guide)

6.9.4.2.2c Effective Area of Circular Tubes and Round HSS

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article specify the effective area, A_{eff}, of circular tubes and round hollow structural sections (HSS) to be used in the provisions of Article 6.9.4.2.2a to compute the nominal compressive resistance, P_n, of these members (see the Discussion of Article 6.9.4.2.2a in this Guide). Circular tubes and round HSS are not used as compression members in the routine steel I-girder bridges covered by this Guide, and so the provisions of this Article are not applicable.

The connection details associated with circular tubes and round HSS members are typically expensive to fabricate.

6.9.4.3 Built-Up Members

6.9.4.3.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article contain general requirements for compression members built-up from rolled or welded shapes connected by continuous plates with or without perforations or by tie plates with or without lacing. Provisions are specified in this Article for the welded or bolted connections between the plates and shapes.

Built-up compression members are not typically used in the routine steel I-girder bridges covered by this Guide, except possibly the application of double-angle members in cross-frames; the provisions of this Article are only applicable in that case. It should be noted that using rolled steel tee (WT) and double-angle sections as cross-frame members for routine steel I-girder bridges is generally discouraged, while the use of single-angle sections is encouraged. The magnitude of the forces in cross-frame members in routine steel I-girder bridges covered by this Guide is generally small enough such that single-angle members are adequate. Cross-frame forces are typically only large enough to warrant the use of rolled steel tee (WT) or double-angle sections in curved and/or skewed steel I-girder bridges. Rolled steel tee (WT) sections are typically quite expensive to fabricate. Tee (WT) sections are cut from full wide-flange (W) shapes and generally require straightening after the cutting process, which adds significant fabrication effort and cost. Double-angle sections are often viewed as problematic from a maintenance perspective; the surfaces between the adjacent angle flanges are difficult or impossible to paint in the field, and/or can suffer from potentially severe pack rust.

For further information on built-up compression members, consult Section 6.6.3.4.4 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge includes an example cross-frame design. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.9.4.3.2 Perforated Plates

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article contains provisions specific to the design of perforated plates used to connect rolled or welded shapes in built-up compression members (see the Discussion of Article 6.9.4.3.1 in this Guide). The routine steel I-girder bridges covered by this Guide do not utilize built-up compression members with perforated plates; therefore, the provisions of this Article are not applicable.

6.9.4.4 Single-Angle Members

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article provides a significantly simplified method for determining the nominal compressive resistance, P_n, of single-angle members subject to combined axial compression and flexure about one or both principal axes. Single angles provide a very economical solution for, and are commonly used as, chord and diagonal members in truss-type cross-frames in routine steel I-girder bridges. Single angles used as cross-frame members in routine steel I-girder bridges must satisfy the following conditions for the provisions of this Article to be applicable:

1. End connections are to a single leg of the angle, and are welded or use a minimum of two bolts
2. The angle is loaded at the ends in compression through the same leg
3. The angle is not subjected to any intermediate transverse loads:

Cross-frame members in routine steel I-girder bridges are typically configured as truss-type structures (K-frames, X-frames, etc.) in a single plane, and are routinely attached either to gusset plates or directly to cross-frame connection plates (stiffeners) using either welded or bolted connections. This is common and economical connection detailing, as can be seen in AASHTO-NSBA Steel Bridge Collaboration's Guidelines G1.4-2006 Guidelines for Design Details and G12.1-2020 Guidelines to Design for Constructability and Fabrication. Cross-frames detailed in accordance with the recommendations in these guidelines can be designed using the provisions of this Article.

As explained in the Commentary: "In essence, these provisions permit the effect of the eccentricities to be neglected when these members are evaluated as axially loaded compression members for flexural buckling only using an appropriate specified effective slenderness ratio, $(K\ell/r)_{eff}$, in place of $(K\ell/r_s)$ in Eq. 6.9.4.1.2-1." See the Discussion of Article 6.9.4.1.2 in this Guide for more information. Eqs. 6.9.4.4-1 or 6.9.4.4-2, as applicable, are typically used to calculate $(K\ell/r)_{eff}$. It is important to note that the length, ℓ, to be used in calculating $(K\ell/r)_{eff}$ is to be taken as the distance between the work points of the end connections measured along the length of the angle, rather than the physical length of the angle member. The resulting value of P_n may need to be adjusted according to the provisions of Article 6.9.4.2.2a if one or both legs of the rolled angle section are classified as slender elements (see the Discussion of Article 6.9.4.2.2a in this Guide).

The effective slenderness ratio indirectly accounts for the bending in the angles due to the eccentricity of the loading allowing the member to be proportioned according to the provisions of Article 6.9.2.1 as if it were a pinned-end concentrically loaded compression member. Furthermore, when the effective slenderness ratio is used, single angles need not be checked for flexural–torsional buckling. (see the Discussion of Article 6.9.4.1.3 in this Guide). It is also important to note that the actual maximum slenderness ratio of the angle member, $K\ell/r_z$, is to be used in checking the limiting slenderness ratio for compression members specified in Article 6.9.3 (see the Discussion of Article 6.9.3 in this Guide) rather than $(K\ell/r)_{eff}$, where K is taken equal to 1.0 for single-angle members (Article 4.6.2.5), ℓ is the actual physical length of the angle member, and r_z is the minimum radius of gyration taken about the minor principal axis of the member.

The Commentary goes on to further explain the basis, and the associated limitations and criteria, for this methodology. This methodology greatly simplifies calculations of the resistance of single-angle members subject to combined axial compression and flexure about one or both principal axes. The calculations can be performed by hand or easily programmed in a spreadsheet.

For further information on single-angle compression members, consult Section 6.6.3.4.5 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge includes an example cross-frame design. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.9.4.5 Plate Buckling Under Service and Construction Loads

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

These provisions do not apply to the design of the routine steel I-girder bridges covered by this Guide.

Post-buckling resistance is assumed at the strength limit state in computing the nominal axial compressive and flexural resistance of noncomposite box-section members with slender webs and/or noncompact or slender flanges or plate panels. Under the provisions of Article 6.9.4.5, these particular elements are investigated to check that plate local buckling does not occur under conditions producing maximum longitudinal compressive stress acting at one or both longitudinal edges of the element under consideration at the service limit state or during construction.

As explained in the Article itself, composite and noncomposite I-section members subject to flexure only, such as are used as main girders in routine steel I-girder bridges, are checked for web bend buckling at various limit states according to the provisions of Article 6.10. Furthermore, the provisions of Article 6.9.4.5 are not applicable to the design of plate elements supported only along one longitudinal edge (such as the flanges of I-shaped members); other provisions of Section 6 of the BDS address the design of flanges in compression for I-shaped members.

6.9.5 Composite Members

6.9.5.1 Nominal Compressive Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of this Article apply to the design of composite columns (i.e., concrete-filled tubes or pipes and concrete-encased shapes) without flexure and are used to determine the nominal compressive resistance, P_n, of these columns. For columns subject to combined axial compression and flexure, the calculation of the nominal flexural resistance, M_n, of concrete-filled tubes or pipes is covered in Article 6.12.2.3.2 and the calculation of M_n of concrete-encased shapes is covered in Article 6.12.2.3.1 (see the Discussion of Articles 6.12.2.3.1 and 6.12.2.3.2 in this Guide). As such, these provisions are considered beyond the scope of superstructure design. These members could potentially be used in substructures for the routine steel I-girder bridges covered by this Guide, although they are not commonly utilized.

6.9.5.2 Limitations

6.9.5.2.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of this Article apply to the design of composite columns (i.e., concrete-filled tubes or pipes and concrete-encased shapes) without flexure and specify limitations that must be met in order to use the nominal compressive resistance equations for these columns specified in Article 6.9.5.1 (see the Discussion of Article 6.9.5.1 in this Guide). As such, these provisions are considered beyond the scope of superstructure design. These members could potentially be used in substructures for the routine steel I-girder bridges covered by this Guide, although they are not commonly utilized.

6.9.5.2.2 Concrete-Filled Tubes

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

This Article applies to the design of composite columns without flexure utilizing concrete-filled tubes or pipes where full composite action is not deemed necessary. These members could potentially be used in substructures for the routine steel I-girder bridges covered by this Guide, although they are not commonly utilized. As such, this provision is considered beyond the scope of superstructure design.

This Article specifies that the wall thickness requirements for unfilled tubes given in Article 6.9.4.2 are to apply to concrete-filled tubes. For applications where full composite action is deemed necessary under combined axial compression and flexure, the provisions of Articles 6.9.6 and 6.12.2.3.3 should be employed instead (see the Discussion of Articles 6.9.6 and 6.12.2.3.3 in this Guide).

6.9.5.2.3 Concrete-Encased Shapes

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of this Article apply to the design of composite columns without flexure utilizing concrete-encased steel shapes and specifies reinforcement requirements for these columns. These members could potentially be used in substructures for the routine steel I-girder bridges covered by this Guide, although they are not commonly utilized. As such, these provisions are considered beyond the scope of superstructure design.

6.9.6 Composite Concrete-Filled Steel Tubes (CFSTs)

6.9.6.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of this Article apply to the design of composite columns utilizing larger-diameter composite concrete-filled steel tubes, or CFSTs, with or without internal reinforcement subject to axial compression or combined axial compression and flexure for non-seismic applications. The computation of the nominal flexural resistance of these members, M_n, as a function of the nominal axial resistance, P_n, is covered in Article 6.12.2.3.3 (see the Discussion of Article 6.12.2.3.3 in this Guide). These members could potentially be used in substructures for the routine steel I-girder bridges covered by this Guide, although they are not commonly utilized. As such, these provisions are considered beyond the scope of superstructure design.

6.9.6.2 Limitations

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of this Article apply to the design of composite columns utilizing larger-diameter composite concrete-filled steel tubes, or CFSTs, with or without internal reinforcement subject to axial compression or combined axial compression and flexure for non-seismic applications. This Article gives specific limitations for the use of these members, including a slenderness ratio limit for the steel tube. These members could potentially be used in substructures for the routine steel I-girder bridges covered by this Guide, although they are not commonly utilized. As such, these provisions are considered beyond the scope of superstructure design.

6.9.6.3 Combined Axial Compression and Flexure

6.9.6.3.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of this Article apply to the design of composite columns utilizing larger-diameter composite concrete-filled steel tubes, or CFSTs, with or without internal reinforcement subject to combined axial compression and flexure for non-seismic applications. This Article deals with the development of a factored stability-based P-M interaction resistance curve for these members. These members could potentially be used in substructures for the routine steel I-girder bridges covered by this Guide, although they are not commonly utilized. As such, these provisions are considered beyond the scope of superstructure design.

6.9.6.3.2 Axial Compressive Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of this Article apply to the design of composite columns utilizing larger-diameter composite concrete-filled steel tubes, or CFSTs, with or without internal reinforcement subject to combined axial compression and flexure for non-seismic applications. This Article covers the calculation of the factored axial compressive resistance, P_r, of these members. These members could potentially be used in substructures for the routine steel I-girder bridges covered by this Guide, although they are not commonly utilized. As such, these provisions are considered beyond the scope of superstructure design.

6.9.6.3.3 Nominal Flexural Composite Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provision in this Article applies to the design of composite columns utilizing larger-diameter composite concrete-filled steel tubes, or CFSTs, with or without internal reinforcement subject to combined axial compression and flexure for non-seismic applications. The provision points to Article 6.12.2.3.3 for the computation of the nominal flexural resistance of these members, M_n, as a function of the nominal axial resistance, P_n (see the Discussion of Article 6.12.2.3.3 in this Guide). These members could potentially be used in substructures for the routine steel I-girder bridges covered by this Guide, although they are not commonly utilized. As such, this provision is considered beyond the scope of superstructure design.

6.9.6.3.4 Nominal Stability-Based Interaction Curve

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of this Article apply to the design of composite columns utilizing larger-diameter composite concrete-filled steel tubes, or CFSTs, with or without internal reinforcement subject to combined axial compression and flexure for non-seismic applications. The provisions of this Article outline the specific steps necessary to modify the material-based interaction curve developed according to the provisions of Article 6.12.2.3.3 (see the Discussion of Article 6.12.2.3.3 in this Guide) to include stability effects based on the buckling load determined in Article 6.9.6.3.2 to create a nominal stability-based interaction curve, which is then multiplied by the appropriate resistance factor specified in Article 6.5.4.2 to determine the final factored resistance of the CFST for combined axial compression and flexure for all load conditions. As such, these provisions are considered beyond the scope of superstructure design. These members could potentially be used in substructures for the routine steel I-girder bridges covered by this Guide, although they are not commonly utilized.

6.10 I-SECTION FLEXURAL MEMBERS

6.10.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article contains general requirements and provisions for I-section flexural design, including a listing of the Articles in Article 6.10 containing the beam or girder proportioning requirements and the design requirements at each limit state, which are typically applicable or at least partially applicable to the girders or beams used as the primary load-carrying members in routine steel I-girder bridges, as described further in the various Discussions of related Articles provided in this Guide.

These provisions would also be applicable (or partially applicable) to an I-section used as the top chord of an end cross-frame, or as an end diaphragm, that is designed as a flexural member to support the wheel loads coming onto the end of the deck.

For routine multi-span continuous rolled-beam bridges, strong consideration should be given to applying the applicable design provisions of the optional Appendix A6 for constructibility and at the strength limit state (see the Discussion of Appendix A6 in this Guide). These provisions account for the ability of some I-sections to develop flexural resistances significantly greater than the yield moment, M_y, when certain proportioning requirements are met; taking advantage of this ability could potentially lead to a much more economical design.

The definition of a routine steel I-girder bridge specifically excludes the use of moment redistribution methods and so the optional provisions of Appendix B6 are considered not applicable to the routine steel I-girder bridges covered by this Guide (see the Discussion of Article 4.6.4.1 in this Guide).

The pitch of the fasteners in the compression and tension flanges of built-up I-section flexural members should satisfy the maximum pitch requirements for stitch bolts in built-up compression members and tension members, respectively, specified in Article 6.13.2.6.3. However, built-up I-section flexural members are not typically used in the routine steel I-girder bridges covered by this Guide.

The discussions in the Commentary for this Article on bridges containing both straight and curved segments, kinked (chorded) girders, the consideration of flange lateral bending effects when cross-frames or diaphragms are placed in discontinuous lines in skewed bridges, and the consideration of flange lateral bending effects in horizontally curved bridges are not applicable to the routine steel I-girder bridges covered by this Guide. For these bridges, flange lateral bending effects tend to be most significant during construction and tend to be insignificant in the final constructed condition.

Flowcharts for flexural design of I-section members according to the provisions of Article 6.10 are provided in Appendix C6. These flowcharts are helpful in guiding the designers through the design provisions at each limit state (see the Discussion of Appendix C6 in this Guide). Fundamental calculations for flexural members (e.g., section property calculations, calculation of the depth of the web in compression, web crippling and web local yielding checks, etc.) are provided in Appendix D6 (see the Discussion of Appendix D6 in this Guide).

For design examples illustrating the flexural design of steel I-girders at each limit state, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The calculations in Examples 2A and 2B illustrating the application of the optional moment redistribution methods specified in Appendix B6 are considered not applicable to the routine steel I-girder bridges covered by this Guide. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

See also AISC/NSBA's Standard Plans for Steel Bridges, NSBA's Span-to-Weight Curves, and the Short Span Steel Bridge Alliance's Technical Design Resources for Short Span Steel Bridges, which provide handy benchmark data for routine steel I-girder bridge designs.

6.10.1.1 Composite Sections

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The Article provides a basic definition of a composite section as a section where the top flange of the steel section is connected to the concrete deck with shear connectors (Article 6.10.10) in the bridge's final, completed condition. The shear connectors transfer horizontal shear from the deck to the girder and prevent slip parallel to the girder between the concrete and the steel. Under these conditions, a linear strain distribution from the top of the deck to the bottom of the girder can be assumed, with a single location of the neutral axis of the section. Under these conditions, the composite concrete deck can also be assumed to provide continuous lateral support to the top flange. This Article is applicable to the routine steel I-girder bridges covered by this Guide (except as noted in the Discussion for Article 6.10.1.2 in this Guide).

Note that composite action is not present during construction, prior to the placement and hardening of the deck.

6.10.1.1.1 Stresses

6.10.1.1.1a Sequence of Loading

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article describes for a composite section the necessary accumulation of the elastic stresses due to the applied dead and live loads acting on different sections; that is, the noncomposite component dead loads (referred to herein as DC_1 loads) acting on the bare steel section, the composite component dead loads (referred to herein as DC_2 loads) acting on the long-term ($3n$) transformed composite section to account for the effects of concrete creep, the wearing surface and utility loads (referred to as DW loads) acting on the long-term ($3n$) transformed composite section, and the live loads plus the dynamic load allowance (LL+IM) acting on the short-term (n) transformed composite section. The calculation of the long-term and short-term transformed composite sections is described in Articles 6.10.1.1.1b and 6.10.1.1.1c (see the Discussion of Articles 6.10.1.1.1b and 6.10.1.1.1c in this Guide). The accumulation of the elastic stresses must be accounted for in the design of steel I-girder bridges at the service and strength limit states (and in some cases involving the dead loads at the fatigue limit state).

This accumulation of the elastic stresses reflects the assumption that the routine steel I-girder bridges covered by this Guide are built using unshored construction, in which no support of the steel beams or girders (other than at permanent support points) is provided during the concrete deck construction, including no temporary supports. As a result, the bare steel beams or girders resist the permanent load applied before the concrete deck hardens and the composite girder section (steel girder alone, steel girder plus the composite concrete deck, or steel girder plus the longitudinal deck reinforcement, as applicable – see the Discussion of Articles 6.10.1.1.1b and 6.10.1.1.1c in this Guide) resists the permanent and transient loads applied after the concrete deck hardens. This reflects the common practices used throughout the United States for the construction of virtually all steel-girder bridges, including the routine steel I-girder bridges covered in this Guide, in which temporary shoring is provided only during steel erection, if at all. Temporary shoring is not provided during the placement of the concrete deck.

Shored construction, in which the steel beams or girders would be theoretically supported along their entire length during the concrete deck construction so that the composite girder would resist both permanent and transient loads, is permitted in this Article but is not recommended and essentially never used since little is currently known about the effects of concrete creep on composite steel girders under large dead loads; therefore, shored construction is considered not applicable to the routine steel I-girder bridges covered by this Guide.

Since plane sections are assumed to remain plane, the calculated elastic stresses at any given point on the cross-section due to the various loadings acting on their associated noncomposite, short-term composite, or long-term composite sections may be summed. However, at elastic stress levels, the principle of superposition does not apply to the bending moments due to the various loadings, as these moments are each applied to different sections (i.e., the noncomposite, short-term composite, and long-term composite sections). The girder stiffness is changing as each moment is applied. Therefore, at elastic stress levels, individual bending moments may not be summed.

Once the yield stress is exceeded however, it is considered acceptable to sum the individual bending moments. This approach would be valid only if the optional provisions of Appendix A6 are being implemented (see the Discussion of Appendix A6 in this Guide).

Several guideline documents, such as the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures and various chapters of the NSBA's Steel Bridge Design Handbook, provide good discussions of this topic. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the stresses in the section in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.1.1.1b Stresses for Sections in Positive Flexure

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article contains provisions for the calculation of flexural stresses for sections in regions of positive flexure and is applicable to the routine steel I-girder bridges covered by this Guide.

For calculating flexural stresses within sections subjected to positive flexure for both long-term and short-term moments applied to the composite section at all limit states, the composite section consists of the steel section and the appropriate transformed area of the effective width of the concrete deck; i.e., the concrete deck is transformed into equivalent steel by dividing the effective width of the deck (see the Discussion of Article 4.6.2.6.1 in this Guide) by the modular ratio. The modular ratio is equal to the ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete (Eq. 6.10.1.1.1b-1). The deck width is reduced rather than the deck thickness to have a less significant effect on the computed moment of inertia.

"Transient loads" consist of live load (LL) and dynamic load amplification, or "impact" (IM). For transient loads (i.e., LL + IM), which are applied to the short-term composite section, the concrete deck area is transformed by using the short-term modular ratio, n.

"Permanent loads" consist of dead loads which are applied to the bridge after the composite deck has been placed and hardened. These loads are grouped into one of two categories. The first category, designated DC_2, represents the weight of structural component and nonstructural attachments which are added to the bridge after deck construction, including such items as barrier rails, medians, attached signs or lights, etc. The second category, designated DW, represents the weight of wearing surfaces that may be applied to the deck (either at the time of initial construction of the bridge or in the future) and the weight of utilities that may be attached to the bridge (either utilities attached at the time of initial construction of the bridge, or a load allowance for utilities that may be attached to the bridge in the future). For DC_2 and DW loads, which are applied to the long-term composite section, the concrete deck is transformed by using the long-term modular ratio, $3n$, to account for the effects of concrete creep. When concrete is placed under a sustained long-term stress, there is an instantaneous elastic strain, followed by a time-dependent increase in strain known as creep. In a composite girder, due to the effects of creep, the strain in the steel girder increases and the steel stresses become larger, while the strains and stresses in the concrete deck are reduced. The short-term modular ratio is based on the initial tangent modulus, E_c, of the concrete, while the long-term modular ratio is based on an effective apparent modulus, $E_c/3$, to account for the effects of concrete creep in an approximate fashion. The effects of creep are usually conservatively ignored in the computation of the stresses in the concrete deck.

The longitudinal deck reinforcement is not considered effective in compression at the strength limit state because it is not tied (i.e., not confined); therefore, its contribution is typically neglected when computing the composite section properties in regions of positive flexure for strength limit state checks. Consideration may be given to including the longitudinal deck reinforcement within the effective width of the deck in the composite section properties when computing the stresses at the fatigue and service limit states. Typically, the area of the concrete deck haunch is not considered in the computation of the composite section properties; the haunch depth may be considered, however, if permitted by the Owner-agency. Note that many Owner-agencies design policies neglect the depth of the haunch in the calculation of section properties since variations from the girder's anticipated camber are typically taken up in the haunch. Also note that the haunch dimension will typically not be a constant value when rolled beam sections are used.

For further discussion and sample calculations of the section properties and elastic stresses for composite sections in regions of positive flexure, consult Sections 6.4.2.3.2 and 6.4.2.4.1 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures, as well as NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

For routine rolled-beam bridges, it is possible that the elastic neutral axis of the transformed composite section may fall within the deck. Concrete on the tension side of the neutral axis is not to be considered effective at the strength limit state; the concrete below the neutral axis is assumed cracked in tension and therefore ineffective. In such cases, the effective transformed area of the concrete becomes a function of the neutral-axis position. Consult pp. 6.194 and 6.195 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge for an example calculation of the composite section properties for such a case.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the stresses in the section in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.1.1.1c Stresses for Sections in Negative Flexure

Determination of applicability, *Simple Span Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Applicable.

Discussion:

Simple Span Bridges:

This Article does not apply for simple span routine steel I-girder bridges as such bridges are only subject to positive moments.

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

This Article contains provisions for the calculation of flexural stresses for sections in regions of negative flexure and is applicable to the multi-span continuous routine steel I-girder bridges covered by this Guide.

For calculating flexural stresses within sections subjected to negative flexure at the strength limit state for both long-term and short-term moments applied to the composite section, the composite section consists of either the steel section alone, or the steel section plus the longitudinal reinforcement within the effective width of the concrete deck if permitted by the Owner-agency policy. The section properties and strength of the concrete deck itself in tension are always ignored at the strength limit state. The AASHTO LRFD BDS permits the longitudinal reinforcement (including the minimum area of longitudinal reinforcement specified for control of deck cracking – Article 6.10.1.7) within the effective width of the deck to be included in the section properties if stud shear connectors are present in regions of negative flexure. Including consideration of the longitudinal reinforcing in the deck can facilitate the use of a smaller top flange than bottom flange in regions of negative flexure and is recommended for the routine steel multi-span continuous I-girder bridges with stud shear connectors in regions of negative flexure if permitted by the Owner-agency.

Articles 6.6.1.2.1 and/or 6.10.4.2.1 permit the concrete deck to be considered effective in tension when computing the composite section properties for negative flexure at the fatigue and/or service limit states, respectively, when certain specified requirements are satisfied (and if permitted by the Owner-agency); otherwise, the concrete deck in tension is ignored (see the Discussion of Articles 6.6.1.2.1 and 6.10.4.2.1 in this Guide). In that case, the properties of the long-term 3*n* composite section (including the transformed area of the concrete deck) would be used for permanent (dead) loads applied after the concrete deck has hardened (i.e., DC_2 and DW loads). The properties of the short-term *n* composite section (including the transformed area of the concrete deck) would be used for transient (live) loads applied after the concrete deck has hardened (i.e., LL+ IM loads). Consideration may be given to including the longitudinal deck reinforcement within the effective width of the concrete deck in the composite section properties in such cases.

For further discussion and sample calculations of the section properties and elastic stresses for sections in regions of negative flexure, consult Sections 6.4.2.3.3 and 6.4.2.4.1 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures, as well as NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder

Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the stresses in the section in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.1.1.1d Concrete Deck Stresses

Determination of applicability, *Simple Span Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Applicable.

Discussion:

Simple Span Bridges:

This Article contains provisions for the calculation of longitudinal flexural stresses in the concrete deck and is conditionally applicable to the routine steel simple span I-girder bridges as described below.

For simple span routine steel I-girder bridges, only the longitudinal compressive stresses in the concrete deck are needed if the section is treated as a noncompact section at the strength limit state (Article 6.10.7.2.1); the concrete deck is not subject to tension in a simple span bridge.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the stresses in the section in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

This Article contains provisions for the calculation of longitudinal flexural stresses in the concrete deck and is applicable to the multi-span continuous routine steel I-girder bridges covered by this Guide.

The longitudinal flexural tensile stresses in the concrete deck are needed to determine the cut-off points for the minimum one-percent longitudinal reinforcement in the deck (see the Discussion of

Article 6.10.1.7 in this Guide) and to determine if the concrete deck may be considered effective in tension at the service limit state (see the Discussion of Article 6.10.4.2.1 in this Guide) in routine steel multi-span continuous I-girder bridges in regions of negative flexure. The longitudinal flexural compressive stresses in the concrete deck are needed to check the specified deck-stress limit for noncompact sections in regions of positive flexure at the strength limit state for routine steel I-girder bridges in which the sections are treated as noncompact sections in these regions (see the Discussion of Article 6.10.7.2.1 in this Guide).

In a composite girder, longitudinal flexural stresses in the concrete deck are assumed to result only from the permanent loads and transient loads applied after the concrete deck has hardened; i.e., the DC_2, DW, and (LL+IM) loads. For reasons described in the Commentary for this Article, the short-term *n*-composite section (including the transformed area of the concrete deck) is to be used to calculate the deck stresses due to these loads. It is important to remember that the calculated stress at the top of the transformed concrete deck must be divided by the modular ratio, *n*, to obtain the maximum stress in the concrete.

For further discussion and a sample calculation of the concrete deck stresses, consult Section 6.4.2.4.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures, as well as NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the stresses in the section in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.1.1.1e Effective Width of Concrete Deck

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article indicates where the provisions are located for the calculation of the effective width of the concrete deck (i.e., Article 4.6.2.6.1), which is used in the calculation of the composite section properties for sections in which the concrete deck is considered effective in compression (or tension, as applicable) for the routine steel I-girder bridges covered by this Guide. Only the tributary width provisions in the first paragraph of Article 4.6.2.6.1 should be considered applicable to the computation of the effective width of the concrete deck for the routine steel I-girder bridges covered by this Guide (see the Discussion of Article 4.6.2.6.1 in this Guide).

6.10.1.2 Noncomposite Sections

Determination of applicability, *Simple Span Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

Simple Span Bridges:

This Article provides the basic definition of a noncomposite section as a section where the top flange of the steel section is not connected to the concrete deck by shear connectors. Thus, slip is assumed to occur between the steel section and the concrete deck. Although unintended composite action does occur, it is conservatively ignored. Therefore, in such cases, flexural stresses in the section due to all permanent and transient loads are computed based on the section properties of the steel section only.

The definition of a noncomposite section is not applicable to the simple span routine I-girder bridges covered by this Guide. Providing no shear connectors along the entire length of the girders/beams of a steel girder bridge is not recommended, and such designs would not represent a routine steel I-girder bridge as covered by this Guide.

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

This Article provides the basic definition of a noncomposite section as a section where the top flange of the steel section is not connected to the concrete deck by shear connectors. Thus, slip is assumed to occur between the steel section and the concrete deck. Although unintended composite action does occur, it is conservatively ignored. Therefore, in such cases, flexural stresses in the section due to all permanent and transient loads are computed based on the section properties of the steel section only.

This Article is only conditionally applicable to multi-span continuous routine steel I-girder bridges covered by this Guide; the provisions of this Article would only apply to sections in which shear connectors are intentionally omitted in regions of negative flexure in multi-span continuous routine steel I-girder bridges. The decision to omit shear connectors in regions of negative flexure is typically dependent on the preferences of the Owner-agency. However, unless specifically required by Owner-agency policy, this practice is not recommended.

6.10.1.3 Hybrid Sections

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article covers the design of hybrid-section members, in which a hybrid section is defined as a fabricated steel section with a web that has a specified minimum yield strength less than one or both flanges. Rolled-beam sections are rolled from a single billet of steel and thus cannot be hybrid sections. The routine steel I-girder bridges covered by this Guide are assumed to contain only homogeneous-section members and do not contain hybrid-section members; therefore, the provisions of this Article are not applicable to their design.

6.10.1.4 Variable Web Depth Members

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article covers the design of variable web depth members, including haunched girders and girders with a linearly varying depth. Rolled-beams sections are always constant depth. The routine steel I-girder bridges covered by this Guide are assumed to contain only constant-depth members; therefore, the provisions of this Article are not applicable to their design.

6.10.1.5 Stiffness

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article contains a description of the stiffness properties for various load types to be assumed in the analysis. The provisions in the first paragraph of this Article are considered applicable to the routine steel I-girder bridges covered by this Guide.

For permanent loads applied to the noncomposite section, the stiffness properties of the steel section alone are to be used in the analysis. For permanent loads and transient loads applied to composite flexural members at all limit states, the stiffness properties of the full composite section are to be used in the analysis, with the stiffness properties of the long-term composite section used for the permanent loads and the stiffness properties of the short-term composite section used for the transient loads. See the Discussions of Article 6.10.1.1.1 and its associated sub-Articles in this Guide for more information on the short-term and long-term composite sections. At sections where the composite stiffness properties are used, the concrete is to be assumed effective in tension and compression for the analysis (i.e., along the entire span length).

In multi-span continuous bridges, it could be theorized that the composite section in negative moment regions would typically have a different stiffness for design calculations at the strength limit state because the concrete deck in tension is assumed cracked for design and not participating. However, moments and deflections computed assuming full composite action agree much better with field measurements than those computed with a assuming no composite action. Consequently, the Article specifies that the concrete deck must be assumed to be effective over the entire span length for the analysis. Assuming the composite stiffness to be effective over the entire span length gives greater girder moments at the pier and slightly smaller mid-span moments compared to analyses based on assuming composite action in the so-called positive moment regions only. The increase in negative girder moments occurs over a relatively short length of what is typically a larger cross-section, while the reduction in moment occurs over a much longer positive moment region.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the section properties for the stiffness analysis in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

The provisions in second paragraph of this Article dealing with the modeling of girder torsional stiffness in skewed and/or curved I-girder bridges are not applicable to the routine steel I-girder bridges covered by this Guide.

6.10.1.6 Flange Stresses and Member Bending Moments

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

General

The first three paragraphs of this Article contain provisions defining the stresses or moments, as applicable, to be used when specific flexural resistance design checks are made. The provisions in these paragraphs are considered applicable to the routine steel I-girder bridges covered by this Guide.

For the checking of lateral-torsional buckling resistance (see the Discussion of Articles 6.10.8.2.3 and A6.3.3 in this Guide), the stress, f_{bu}, is to be determined as the value of the flange compressive stress at the cross-section where $f_{bu}/R_bR_hF_{yc}$ is maximum in the unbraced length under consideration, including the end cross-sections, calculated without consideration of flange lateral bending. R_b is the *minimum* web load-shedding factor within the unbraced length under consideration, including the end cross-sections (see the Discussion of Article 6.10.1.10.2 in this Guide). R_h is the hybrid factor (see the Discussion of Article 6.10.1.10.1 in this Guide) taken equal to 1.0 since the routine steel I-girder bridges covered by this Guide do not utilize hybrid-section members. The moment, M_u, is to be determined as the value of the major-axis bending moment at the cross-section where $M_u/R_{pc}M_{yc}$ is maximum in the unbraced length under consideration, including the end cross-sections. R_{pc} is the web plastification factor for the compression flange at the cross-section under consideration determined as specified in Article A6.2.1 or A6.2.2, as applicable (see the Discussion of Articles A6.2.1 and A6.2.2 in this Guide). M_{yc} is the yield moment with respect to the compression flange at the cross-section under consideration determined as specified in Article D6.2 (see the Discussion of Article D6.2 in this Guide).

For design checks where the flexural resistance is based on yielding, flange local buckling or web bend-buckling, f_{bu} and M_u may be determined as the corresponding values at the section under consideration.

This Article also contains provisions related to flange lateral bending stresses due to torsion and the amplification of such stresses in discretely braced compression flanges. A discretely braced flange is defined as a flange supported at discrete intervals by bracing sufficient to restrain lateral deflection of the flange and twisting of the entire cross-section at the brace points (cross-frames or diaphragms, see the Discussion of Article 6.7.4 in this Guide).

Sources of Flange Lateral Bending

Flange lateral bending stresses due to primary dead load and live load effects are not significant in routine steel I-girder bridges as defined for the purposes of this Guide, since their geometric parameters and framing do not lend themselves to the development of significant torsion in the girders. Instead, for the routine steel I-girder bridges covered by this Guide, flange lateral bending stresses generally arise from the following two sources:

- Torsion due to deck overhang loads acting on the discretely braced flanges of the bare steel exterior (fascia) girders in regions of positive flexure during the concrete deck construction; and
- Flange lateral bending stresses due to wind loads, both during construction and in the final condition.

For more information on deck overhang loads during construction and the torsion they cause in girders, please see the Discussion of Article 6.10.3.4.1 in this Guide.

Wind loading is a consideration both during construction and in the final condition and should be addressed in the girder constructibility checks. During construction, wind loading contributes to flange lateral bending stresses potentially both before and during deck placement. Typically, a higher wind velocity (and thus greater wind load) is assumed during “construction inactive” conditions (i.e., when the structural steel is erected, but prior to deck placement operations, such as for example during overnight or weekend times, or during storm events, when the contractor is not working) than is assumed during “construction active” conditions (i.e., during the time when the contractor is actively working to place the concrete deck). Appropriate load combinations should be developed and investigated, considering both permanent and temporary dead loads, wind load, and the presence or absence of construction live loads (see the Discussion of Article 3.4.2.1 in this Guide). Many Owners have specific policies regarding required loads and load combinations to investigate during construction. The AASHTO *Guide Specification for Wind Loads on Bridges During Construction* (see the Discussion of Article 4.6.2.7.3 in this Guide) and the Reference Manual for NHI Course 130102, Engineering for Structural Stability in Bridge Construction also provide good discussion of wind loading during construction. Under final conditions (i.e., when construction is complete and the bridge is in service), wind loads acting on discretely braced bottom flanges cause flange lateral bending (i.e., at the strength limit state). In lieu of a more refined analysis, Article C6.10.3.4.1 gives approximate equations for calculation of the maximum flange lateral bending moments due to eccentric concrete deck overhang loads (see the Discussion of Article 6.10.3.4.1 in this Guide). Determination of flange wind moments is addressed in Article 4.6.2.7 (see the Discussion of Article 4.6.2.7 in this Guide).

Treatment of Flange Lateral Bending Stresses

It should be noted that most line girder analysis programs do not directly calculate flange lateral bending stresses. Line girder analysis programs use one-dimensional analysis models which cannot directly address the torsional loading effects that produce flange lateral bending. Some programs include the ability for the user to manually input flange lateral bending moments or stresses, calculated by the user outside the program; other programs may not have this ability. In many cases, it may be easiest to evaluate the effects of flange lateral bending on the design outside of a line girder analysis program using hand calculations, perhaps facilitated by programming these calculations in a spreadsheet. This allows the designer flexibility, particularly in addressing the various constructibility checks which must be performed. If it turns out that consideration of the effects of flange lateral bending, whether in a constructibility check, or in the supplemental checks of the bottom flange under final conditions, results in a controlling design condition, the designer may have to rerun the line girder analysis with resized girders to update major-axis bending stress calculations, deflection calculations, or other calculations.

For discretely braced compression flanges, the largest lateral bending stress, f_ℓ, throughout the unbraced length of the flange must be used in combination with f_{bu} or M_u when the flexural resistance is based on lateral-torsional buckling; otherwise, f_ℓ may be determined as the corresponding value at the section under consideration. When the maximum values of f_ℓ and f_{bu} or M_u occur at different locations within the unbraced length, it is conservative to use the maximum values in a single application of the yielding and flange local buckling resistance equations. Flange lateral bending is not a consideration in the web bend-buckling resistance equations because the flange lateral bending stress is zero at the web. Top flange lateral bending stresses are ignored once the flange is continuously braced by the hardened concrete deck. The resistance of the composite concrete deck is adequate to compensate for the neglect of these initial lateral bending stresses.

The upper limit on the lateral bending stresses in discretely braced flanges given by Eq. 6.10.1.6-1 applies to the lateral bending stresses in routine steel I-girder bridges. The limit applies after any necessary amplification is applied to the first-order lateral bending stresses (see below). For cases in which the total elastically-computed flange lateral bending stress is larger than the limit of $0.6F_{yf}$, the reduction in the major-axis bending resistance due to flange lateral bending tends to be greater than that determined based on the approximate one-third rule flexural resistance equations utilized throughout Section 6 to combine the effects of major-axis bending stresses and flange lateral bending stresses due to torsion. For further information on the one-third rule flexural resistance equations and their development, consult Section 6.5.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

Amplification of First-Order Flange Lateral Bending Stresses

To determine whether amplification of the first-order lateral bending stress, $f_{\ell 1}$, in a discretely braced compression flange is required in routine steel I-girder bridges, Eqs. 6.10.1.6-3 and 6.10.1.6-5 apply only when checking bottom-flange lateral bending stresses due to wind load in the final condition (i.e., at the strength limit state) in regions of negative flexure in beams or girders designed using the provisions of the optional Appendix A6 (see the Discussion of Appendix A6 in this Guide); otherwise, Eqs. 6.10.1.6-2 and 6.10.1.6-4 apply.

If the unbraced length, L_b, or distance between the brace points exceeds the limit given by the applicable Eq. 6.10.1.6-2 or 6.10.1.6-3 (Note: refer to the "where" list underneath these equations for the proper definition of the terms to be used in each of these equations), which is typically the case, the second-order flange lateral bending stress may be approximated by amplifying the first-order value by the amplification factor given by either Eq. 6.10.1.6-4 or 6.10.1.6-5, as applicable. Note that the elastic lateral-torsional buckling stress, F_e, is not limited to $R_bR_hF_{yc}$ in the computation of the amplification factor. Also, for unbraced lengths where Article 6.10.8.2.3 is applied (see the Discussion of Article 6.10.8.2.3 in this Guide), f_{bu} in the amplification factor Eq. 6.10.1.6-4 is to be taken as the value of the flange compressive stress at the cross-section where $f_{bu}/R_bR_hF_{yc}$ is maximum in the unbraced length under consideration, including the end cross-sections, calculated without consideration of flange lateral bending. For unbraced lengths where Article A6.3.3 is applied (see the Discussion of Article A6.3.3 in this Guide), M_u in the amplification factor Eq. 6.10.1.6-5 is to be taken as the value of the major-axis bending moment at the cross-section where $M_u/R_{pc}M_{yc}$ is maximum in the unbraced length under consideration, including the end cross-sections. The largest value of $f_{\ell 1}$ within the unbraced length under consideration is to be used in the computation of the amplification factor when the flexural resistance is based on lateral-torsional buckling; otherwise, $f_{\ell 1}$ may be determined as the corresponding value at the section under consideration.

When the amplification factor is large (e.g., greater than about 2.5), the flange is likely too narrow; increasing the width of the top flange should reduce the amplification factor to a more reasonable value. Amplification of the first-order lateral bending stresses in discretely braced tension flanges is not required.

The language in the Commentary for this Article dealing with the amplification of first-order flange lateral bending stresses in horizontally curved I-girders (along with Figure C6.10.1.6-1) does not apply to the routine steel I-girder bridges covered by this Guide, which use straight (tangent) girders by definition.

6.10.1.7 Minimum Negative Flexure Concrete Deck Reinforcement

Determination of applicability, *Simple Span Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Applicable.

Discussion:

Simple Span Bridges:

The minimum one-percent longitudinal reinforcement in the deck specified in this Article is not required for crack control in simple span routine I-girder bridges because the concrete deck is in compression; temperature and shrinkage reinforcement is still required however.

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

This Article deals with the minimum required longitudinal concrete deck reinforcement in regions of negative flexure for control of deck cracking and is applicable to the routine steel multi-span continuous I-girder bridges covered by this Guide.

The Article prescribes the total cross-sectional area of the longitudinal reinforcement (including the temperature and shrinkage reinforcement) that is provided in these regions, the maximum reinforcing bar size and required yield strength, and the layout, distribution, and maximum spacing of the reinforcing. The reinforcement is to be distributed across the entire concrete section since the effective width of concrete deck is close to the entire deck width in most girder bridges. The use of small bars at relatively close spacing (not exceeding 12 inches) is intended to result in closely spaced cracks of small width. Applicable Owner-agency standards should always be consulted for any Owner-specific size, placement, and spacing requirements.

The required cross-sectional area of longitudinal reinforcement is a function of the cross-sectional area of the cast-in-place portion of the deck concrete. The Article requires that the cross-sectional area of longitudinal reinforcement shall not be less than one percent of the "cross-sectional area of the concrete deck," and clarifies that when partial-depth precast concrete deck panels are used the calculation of the cross-sectional area of the longitudinal reinforcing shall be based on the cross-sectional area of the cast-in-place portion of the deck only.

The detailing of the longitudinal reinforcing is also affected by whether or not partial-depth precast concrete deck panels are used. When the full depth of the concrete deck is cast-in-place, the longitudinal reinforcing should be placed in two layers uniformly distributed across the width of the deck, with two-thirds of that reinforcing placed in the top layer. In practical terms, it is generally not possible to place exactly two-thirds of the longitudinal reinforcing in the top layer and exactly one-third in the bottom layer; the Commentary clarifies this by saying, "Where feasible, approximately two-thirds of the required reinforcing should be placed in the top layer." In cases where the detailing results in the top layer of reinforcing providing slightly more than two-thirds of the one percent reinforcing ratio requirement, it is not necessary to also provide more than one-third of the one percent reinforcing ratio in the bottom layer; it is only necessary that the total cross-sectional area of the longitudinal reinforcing meet the one percent reinforcing ratio requirement.

Conversely, when partial-depth precast concrete deck panels are used, the cast-in-place portion of the deck is usually only thick enough to accommodate a single layer of longitudinal reinforcing in a practical manner; the provisions of the Article and the Commentary reflect this practical detailing issue.

For the computation of girder section properties for the strength limit state checks in regions of negative flexure in which the longitudinal reinforcement is considered to act with the steel section, the area of the reinforcement in the two layers within the effective width of the deck over the beam or girder under consideration can be combined into a single layer placed at the centroid of the two layers, if desired.

The minimum one-percent longitudinal reinforcement in the deck is to be placed along the span wherever the longitudinal tensile stress in the concrete deck (see the Discussion of Article

6.10.1.1.1d in this Guide) due to either the factored construction loads (i.e., during the sequential deck placement) or due to load combination Service II (Table 3.4.1-1) exceeds $0.9f_r$, where f_r is the modulus rupture of the concrete taken equal to $0.24\sqrt{f'_c}$ for the routine steel I-girder bridges covered by this Guide. f'_c may be taken as the 28-day compressive strength of the concrete or else a more accurate estimate of the concrete strength at the time the deck casts are made can be used to compute f_r and the modular ratio for this check, if desired.

Many routine steel I-girder bridges will have their decks placed following a sequential deck placement scheme. In this scenario, the deck is placed in series of longitudinal sections, with the maximum size of a section typically limited by the maximum volume of concrete that can be placed at a time. The specific dimensions and limits of each section are also determined to try to approximately correspond to the points of permanent load contraflexure. The order of placement of concrete in each section should generally be specified so that positive moment regions are placed first and negative moment regions are placed last; this helps to minimize the introduction of tensile stresses in the deck in the negative moment regions. The load factor for investigation of the deck stress during a sequential deck placement is taken equal to 1.4 (see the Discussion of Article 3.4.2.1 in this Guide). During the sequential deck placement, when the concrete deck is placed in a span adjacent to a span where the concrete has already been placed, negative moment in the adjacent span causes tensile stresses in the previously placed concrete although these regions may be subjected primarily to positive flexure in the final condition. This requirement is intended to help control cracking in the previously placed concrete.

After the deck hardens, the deck can experience significant tensile stresses outside the points of permanent load contraflexure under moving live loads; the Service II requirement is intended to help control the deck cracking in such cases under expected severe traffic loadings (see the Discussion of Article 6.10.4.2 in this Guide for further discussion on the Service II load combination).

Satisfaction of the above provisions, along with the provision of shear connectors along the entire length of the span, allows the concrete deck to be considered effective in tension when computing the composite section properties for negative flexure at the fatigue and/or service limit states if permitted by the Owner-agency (see the Discussion of Articles 6.6.1.2.1 and 6.10.4.2.1 in this Guide).

The discussion related to the checking of nominal yielding in the longitudinal reinforcement under load combination Service II given in the Commentary for this Article does not apply to the routine steel I-girder bridges covered by this Guide.

The requirement to provide sufficient development length of the longitudinal reinforcement given in the last paragraph of the specification in this Article is only applicable in cases where shear connectors are intentionally omitted in regions of negative flexure, which is dependent on the preferences of the Owner-agency but is not recommended.

Beyond the limits of where the minimum one-percent longitudinal reinforcement in the deck is required (i.e., in regions of the deck where the tensile stress in the deck is below the above-cited limits), temperature and shrinkage reinforcement is still required in the deck.

For further discussion and sample calculations of the minimum negative flexure concrete deck reinforcement, consult Section 6.4.2.3.3.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures, as well as NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.10.1.8 Tension Flanges with Holes

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article provides a limit on the maximum factored major-axis bending stress permitted on the gross section of the beam or girder at the strength limit state or during construction, neglecting the loss of area due to holes in the tension flange.

This Article is applicable to routine steel I-girder bridges at bolted field splices. Eq. 6.10.1.8-1 will prevent a bolted field splice from being located at a section where the factored flexural resistance of the section at the strength limit state exceeds the moment at first yield, M_y, unless the factored stress in the tension flange at that section is limited to the value given by the equation; this may dictate the location of the bolted field splices in some simple span routine steel bridges and in some longer-span multi-span continuous bridges, or it may force an increase in sizing of the flanges of a steel plate girder or an increase in overall sizing of a rolled beam in order to reduce the stresses. However, this equation typically does not control the design for most multi-span continuous bridges with reasonably well-balanced spans where the bolted field splices can be located at or near the points of permanent load contraflexure.

This equation can also control where cross-frame or diaphragm connection plates are attached to the tension flange by bolting (instead of welding), although the use of a bolted connection in this situation is not recommended (see the Discussion of Articles 6.6.1.2.3 and 6.6.1.3.1 in this Guide), and also where lateral bracing members or lateral connection plates are bolted directly to the tension flange, which is a recommended detail for such members to provide improved fatigue performance. However, the routine steel I-girder bridges covered by this Guide are assumed not to contain any lateral bracing.

6.10.1.9 Web Bend-Buckling Resistance

6.10.1.9.1 Webs without Longitudinal Stiffeners

Determination of applicability, *Simple Span Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

Simple Span Bridges and *Multi-span Continuous Plate Girder Bridges*:

The computation of the theoretical web bend-buckling resistance, F_{crw}, is only applicable during construction for simple span routine I-girder bridges utilizing slender web sections in the noncomposite condition. Therefore, these provisions are conditionally applicable for the simple span routine bridges covered by this Guide; that is, the provisions are applicable during construction in the noncomposite condition for simple span bridges utilizing steel plate girders with slender webs, but are not applicable for simple span bridges utilizing steel plate girders with nonslender webs or for simple span bridges utilizing rolled beams (assuming all sections qualify as compact web or noncompact web sections in the noncomposite condition during construction); the reasons for this are discussed further below. See the Commentary for Article 6.10.6.2.3 and the Discussion of Article 6.10.6.2.3 in this Guide for further discussion on the definition and categorization of compact web, noncompact web, and slender web sections.

The computation of F_{crw} is only applicable for routine multi-span continuous plate girder bridges utilizing slender web sections in the noncomposite condition during construction, and/or utilizing slender web sections in regions of negative flexure at the service limit state. In the computation of F_{crw} for composite sections in negative flexure at the service limit state, when the necessary conditions are satisfied such that the concrete deck may be considered effective in tension (see the Discussion of Article 6.10.4.2.1 in this Guide), the depth of the web in compression in the elastic range, D_c, must be computed using Eq. D6.3.1-1 in Appendix D6 (see the Discussion of Article D6.3.1 in this Guide). Otherwise, D_c is to be computed using the steel section alone or the steel section plus the longitudinal reinforcement depending on the preferences of the Owner-agency. Therefore, these provisions are conditionally applicable for the routine multi-span continuous plate girder bridges covered by this Guide.

This Article presents the provisions necessary to compute F_{crw} for webs without longitudinal stiffeners, which is used as a simple index to control strains and transverse displacements in the compression zone of slender-web girders during construction (see the Discussion of Article 6.10.3.2.1 in this Guide) and in regions of negative flexure at the service limit state (see the Discussion of Article 6.10.4.2.2 in this Guide). F_{crw} is not needed at the fatigue limit state because the web bend-buckling check at the service limit state will always control.

The advent of composite design has led to a significant reduction in the size of compression flanges in regions of positive flexure. As a result, more than half of the web of the noncomposite section will be in compression in these regions during the construction condition before the concrete deck has hardened. Slender web sections are more susceptible to bend-buckling in this condition. At the service limit state, a control on the amount of transverse web displacement is also desirable. In a multi-span continuous girder at the service limit state, slender web sections in regions of negative flexure are most susceptible to web bend-buckling, especially for composite sections when the necessary conditions are satisfied such that the concrete deck may be considered effective in tension (see the Discussion of Article 6.10.4.2.1 in this Guide).

For a compact web section (e.g., a rolled-beam section) or a noncompact web section, F_{crw} will always equal or exceed F_{yc}, where F_{yc} is the specified minimum yield strength of the compression flange; therefore, theoretical web bend-buckling in these sections will not occur for elastic stress levels, computed according to beam theory, smaller than the limit of their flexural resistance. Therefore, for these sections, the web bend-buckling checks described above need not be made. The web bend-buckling checks also need not be made for sections in regions of positive flexure at the service limit state in the routine I-girder bridges covered by this Guide (see the Discussion of Article 6.10.4.2.2 in this Guide); however, these regions must be checked for web bend-buckling during construction in routine plate-girder bridges utilizing slender web sections in the noncomposite condition.

F_{crw} is to be checked against the maximum factored compression flange major-axis bending stress. Utilizing the maximum compressive stress in the web rather than the stress in the compression flange to obtain greater precision is not warranted for this check.

The bend-bucking coefficient of 7.2 mentioned at the end of this Article is only applicable around points of permanent-load contraflexure in multi-span continuous bridges and applies to rare cases where both the top and bottom edges of the web are subject to small accumulated compressive stresses.

For further discussion and sample calculations of F_{crw}, consult Section 6.4.5.5 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

Multi-span Continuous Rolled Beam Bridges: the computation of F_{crw} is not applicable for routine multi-span continuous rolled beam bridges during construction or at the service limit state (assuming all sections qualify as compact web or noncompact web sections in the noncomposite condition during construction and in regions of negative flexure at the service limit state, which should typically be the case).

6.10.1.9.2 Webs with Longitudinal Stiffeners

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article presents the provisions necessary to compute the theoretical web bend-buckling resistance, F_{crw}, for webs with longitudinal stiffeners. These provisions are not applicable to the routine steel I-girder bridges covered by this Guide, which do not utilize web longitudinal stiffeners.

6.10.1.10 Flange-Strength Reduction Factors

6.10.1.10.1 Hybrid Factor, R_h

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article presents provisions to compute the hybrid factor, R_h, which is a flange-stress reduction factor that accounts for the redistribution of stress from the web to both flanges resulting from local yielding of the web in hybrid-section members. This Article is not applicable to the routine steel I-girder bridges covered by this Guide, which do not utilize hybrid-section members; therefore, R_h should always be taken equal to 1.0 for these bridges.

6.10.1.10.2 Web Load-Shedding Factor, R_b

Determination of applicability, *Simple Span Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

The Article presents provisions to compute the web load-shedding factor, R_b, which is a post-buckling flange-stress reduction factor that accounts for the nonlinear variation of stresses subsequent to local bend-buckling of slender webs at the strength limit state. The factor accounts for the reduction in the section flexural resistance caused by the shedding of the compressive stresses in the web resulting from local bend-buckling of a slender web at the strength limit state and the corresponding increase in the flexural stress within the compression flange. The R_b factor is not applied in determining the nominal flexural resistance of the tension flange at the strength limit state since the tension flange stress is not increased significantly by the shedding of the web compressive stresses.

Simple Span Bridges:

Web bend-buckling is not a consideration for compact web sections (e.g., rolled beam sections) or noncompact web sections at any limit state (see the Discussion of Article 6.10.1.9.1 in this Guide). Web bend-buckling is explicitly prevented during construction for plate girder sections with slender webs (see the Discussion of Article 6.10.3.2.1 in this Guide) and is not a consideration for composite plate girder sections in regions of positive flexure without web longitudinal stiffeners at the service or strength limit states. Therefore, load-shedding from the web to the compression flange does not theoretically occur in either case and these provisions are not applicable for the routine simple span I-girder bridges covered by this Guide (i.e., R_b is always taken equal to 1.0).

See the Commentary for Article 6.10.6.2.3 and the Discussion of Article 6.10.6.2.3 in this Guide for further discussion on the definition and categorization of compact web, noncompact web, and slender web sections.

Multi-span Continuous Rolled Beam Bridges:

Web bend-buckling is not a consideration for compact web sections (e.g., rolled beam sections) in any regions of the beam at any limit state (see the Discussion of Article 6.10.1.9.1 in this Guide); therefore, load-shedding from the web to the compression flange does not theoretically occur and these provisions are not applicable for the routine multi-span continuous rolled beam bridges covered by this Guide (i.e., R_b is always taken equal to 1.0).

See the Commentary for Article 6.10.6.2.3 and the Discussion of Article 6.10.6.2.3 in this Guide for further discussion on the definition and categorization of compact web, noncompact web, and slender web sections.

Multi-span Continuous Plate Girder Bridges:

Web bend-buckling is not a consideration for compact web plate girder sections or noncompact web plate girder sections at any limit state (see the Discussion of Article 6.10.1.9.1 in this Guide). Web bend-buckling is explicitly prevented during construction for plate girder sections with slender webs (see the Discussion of Article 6.10.3.2.1 in this Guide) and is not a consideration for composite plate girder sections in regions of positive flexure without web longitudinal stiffeners at the service or strength limit states. Thus, the R_b factor will be less than 1.0 only for slender web sections in negative flexure at the strength limit state and therefore these provisions are conditionally applicable for the routine multi-span continuous plate girder bridges covered by this Guide.

See the Commentary for Article 6.10.6.2.3 and the Discussion of Article 6.10.6.2.3 in this Guide for further discussion on the definition and categorization of compact web, noncompact web, and slender web sections.

For the routine multi-span continuous plate girder bridges covered by this Guide, in cases where the R_b factor is less than 1.0 and must be computed (indicating that web bend-buckling has theoretically occurred at the strength limit state), only Eqs. 6.10.1.10.2-3, 6.10.1.10.2-5, 6.10.1.10.2-8, and 6.10.1.10.2-9 are applicable. The other equations in this Article are only applicable to sections with web longitudinal stiffeners and the routine steel I-girder bridges covered by this Guide do not contain web longitudinal stiffeners. Also, the R_b factor must only be computed at the strength limit state for sections in regions of negative flexure that do not satisfy Eq. 6.10.1.10.2-1 (i.e., slender web sections); otherwise, R_b is taken equal to 1.0.

For further discussion and sample calculations of the R_b factor, consult Section 6.4.5.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that tthese references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.10.2 Cross-Section Proportion Limits

6.10.2.1 Web Proportions

6.10.2.1.1 Webs without Longitudinal Stiffeners

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article provides web proportioning limits for sections without web longitudinal stiffeners and is applicable to the routine steel I-girder bridges covered by this Guide.

By limiting the maximum web slenderness of girders without longitudinal stiffeners to 150 as specified in Eq. 6.10.2.1.1-1, transverse stiffeners need only be provided for shear in the routine steel I-girder bridges covered by this Guide and can potentially be spaced up to the maximum specified limit of 3*D* (i.e., three times the web depth).

The AASHTO-NSBA Steel Bridge Collaboration Guideline G12.1-2020 Guidelines to Design for Constructability and Fabrication provides practical guidance for economical proportioning of girder webs, such as the general preference of fabricators for a minimum web thickness of ½" to reduce the deformation of the web and the potential for weld defects during fabrication.

Changes in the web thickness along the girder in plate-girder bridges preferably should be made at field splices. In field sections over interior piers in continuous spans, the web thickness may have to be increased (typically in 1/16-inch increments) over the thickness provided in adjacent regions in positive flexure in some instances; e.g., if the web bend-buckling resistance is exceeded in regions of negative flexure at the service limit state (see the Discussion of Articles 6.10.1.9.1 and 6.10.4.2.2 in this Guide).

A useful guideline for determining the trade-off between adding more transverse stiffeners versus increasing the thickness of the web material in routine plate-girder bridges is that approximately 4 to 5 pounds of web material should be saved for every 1 pound of stiffener material added. This higher unit cost reflects that additional fabrication effort is required per pound of stiffener still than per pound of girder web steel. Generally, an unstiffened web is not the most economical alternative for a plate-girder bridge. The best solution usually includes a limited number of transverse stiffeners over the piers and near the abutments; a so-called partially stiffened web. Transverse stiffeners (other than connection plates for cross-frames or diaphragms) are typically not required in routine rolled-beam bridges.

The web depth of a plate girder dictates the flange sizes. In the absence of depth restrictions or significantly unbalanced spans, the web depth that is selected should be near the optimum web depth for the largest span in the unit, which is the web depth that provides the minimum cost girder. In multi-span continuous bridges, the optimum web depth for the regions of negative flexure is often not the same as that for the regions in positive flexure. Usually, the optimum depth for the regions of positive flexure is a better choice when combined with heavier flanges in the shorter regions of negative flexure. Where a deeper web in the regions of positive flexure requires smaller flanges, this may lead to stability issues during shipping and erection. A compromise depth is usually necessary.

The optimum web depth for a composite girder is elusive since loads are applied to different sections; there is no single algorithm that gives the optimum web depth. Instead, the optimum web depth is best established by preparing a series of designs with different web depths to arrive at an optimum cost-effective depth based on weight and/or cost. The NSBA's LRFD Simon line-girder analysis and design program contains a useful web-depth optimization option that automatically generates a series of trial-design input files from an acceptable starting design input file; the generated input files differ from the starting input only in the vertical web depth. Each of these input files is processed automatically by the analysis engine, and a table is prepared listing the depth, weight, and cost (based on user-input cost factors) for the depths that have acceptable designs. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

For further information on girder depth and I-girder sizing and proportioning, consult Sections 6.3.3.2 and 6.3.4 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.10.2.1.2 Webs with Longitudinal Stiffeners

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides web proportioning limits for sections with web longitudinal stiffeners and is not applicable to the routine steel I-girder bridges as defined for the purposes of this Guide since these bridges do not utilize web longitudinal stiffeners.

6.10.2.2 Flange Proportions

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

Sizing of flanges is one of the most important issues in obtaining an economical steel plate-girder bridge. This Article provides four separate flange proportioning limits that are applicable to the routine steel I-girder bridges covered by this Guide. The reasoning behind each of these limits is discussed in the Commentary. The limits apply to both the compression and tension flanges. Eq. 6.10.2.2-1 in particular provides an upper bound limit for flange slenderness and should not necessarily be used as an indication of an economical design. Eq. 6.10.2.2-2 provides a lower bound limit on the flange width; a larger flange width will often be required.

Fabricators prefer that plate-girder flange widths not be less than 12 inches to avoid distortion and cupping of the flanges during welding, which sets a practical lower limit on the width. According to the AASHTO-NSBA Steel Bridge Collaboration Guideline G12.1-2020 Guidelines to Design for Constructability and Fabrication, fabricators prefer that flange thicknesses not be less than ¾ inches.

Meeting the top flange $L_{fs}/85$ minimum width guideline presented in Eq. C6.10.2.2-1 tends to result in individual girder field sections that are more stable and easier to handle during lifting, erection, and shipping without the need for special stiffening trusses or falsework. This parameter should be checked in conjunction with the flange proportioning requirements of this Article to establish a minimum required top-flange width for each individual unspliced girder field section in the bridge.

Efficient location of flange thickness transitions at shop-welded splices in plate-girder flanges is a matter of plate length availability and the economics of welding and inspecting a splice compared to the cost of extending a thicker plate. The design plans should consider allowing the option to eliminate or move a shop splice by extending a thicker flange plate, subject to the approval of the Engineer. When evaluating such a request, the Engineer should consider the effect of the thicker plate on the girder deflections and stresses. Usually, a change in plate length does not significantly affect the deflections as much as the removal of a welded splice.

Parameters affecting the cost of shop-welded splices vary from shop to shop. Table 1.5.4-1 in the AASHTO-NSBA Steel Bridge Collaboration G12.1-2020 Guidelines to Design for Constructability and Fabrication gives suggested weight savings per inch of flange width to help evaluate placement of shop splices. Usually, somewhere between 800 to 1,200 pounds of material must be saved to justify the introduction of a welded shop splice. This may vary so consider consulting the fabricator regarding this issue whenever possible. Pricing considerations dictate that optimal ordered plate lengths are usually less than or equal to 80 feet. This is also the length that usually fits on a single railroad flat car. Longer plates may of course be used as necessary. Table 1.4.1-1 of the AASHTO-NSBA Steel Bridge Collaboration Guideline G12.1-2020 Guidelines to Design for Constructability and Fabrication provides a table of maximum plate length availabilities for ASTM A 709 Grades 36, 50, and 50W plates. Table 1.4.2-1 provides maximum wide flange beam length availability.

In typical cases, providing more than two shop splices (i.e., three different flange thicknesses) in any one field section of a plate girder is not economical, except in relatively rare cases where the girders are unusually heavy, or plate length availability limits dictate the need for additional splices with or without a thickness change. At flange shop splices, the area of the thinner plate should not be less than one-half the area of the thicker plate to reduce the stress concentration and produce a smoother transition of stress across the splice.

As a practical matter, fabricators typically order plate for flange material from the mills in widths 60 inches and above; typically, the most economical plate size to buy from a mill is between 72 and 96 inches. Thus, consider sizing flanges so that as many pieces as possible can be obtained from a wide plate with minimal waste. To minimize waste, it is also important to limit the number of different flange plate thicknesses specified for a given project. Larger order quantities of plate cost less and minimizing the number of different thicknesses simplifies fabrication and inspection and reduces mill quantity extras. Also, it is preferred to select flange thicknesses in at least 1/8 inch increments up to 2 ½ inches and in ¼ inch increments over 2 ½ inches, and to limit flange thicknesses to 3 inches or below if possible.

Flange widths for an individual plate girder should be kept constant within each field section; i.e., avoid changing flange widths at welded shop splices. Change the flange widths at a bolted field splice instead. There is little need to maintain a constant flange width between individual field sections. However, some Owners may prefer a constant-width bottom flange along the entire length of the girder for aesthetic reasons if pedestrians or vehicles are expected to pass underneath the bridge. Note that top and bottom flange widths within a field section can be, and often are, different.

For further information on sizing flanges for efficient fabrication, consult Section 1.5 of the AASHTO-NSBA Steel Bridge Collaboration Guideline G12.1-2020 Guidelines to Design for Constructability and Fabrication and Section 6.3.4.4.5 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.10.3 Constructibility

6.10.3.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The provisions of this Article cover the required design checks for constructibility, which apply to the routine steel I-girder bridges covered by this Guide.

For routine steel I-girder design, the constructibility checks apply to the timeframe prior to when the concrete deck is cured (i.e., the final structural condition).

The standard of care in many jurisdictions is that the designer need only perform a non-structural review of the conceptual erection sequence for the structural steel framing, primarily to demonstrate that a viable erection scheme exists (i.e., an erection sequence that is feasible given the known site conditions and constraints, specified maintenance-of-traffic sequence and requirements, etc.), including consideration of the location of shoring towers, lifting and holding cranes, etc. Owner-agencies in these jurisdictions expect detailed erection engineering to be performed by the Contractor’s engineer, not by the bridge’s designer. However, several Owner-agencies do require that the designer perform some level of detailed erection engineering. Review local Owner-agency design policies and construction specifications and the local standard of care to determine the requirements in any given specific jurisdiction. Note that the performance of detailed erection engineering is beyond the scope of this Guide.

However, once the structural steel framing system is fully erected, the designer clearly has responsibility for checking that the structural steel has sufficient strength and stiffness to resist construction loads. The provisions outlined in this Article generally apply to the checking of the combined effects of the component dead loads (DC_1 loads) acting on the bare steel girders during construction, including consideration of the deck-placement sequence for multi-span continuous bridges, the deck overhang loads, as applicable (see the Discussion of Article 6.10.3.4.1 in this Guide for more information on deck overhang loads), construction live loads (due to the presence of construction workers, a deck screed machine, and possibly other equipment), and wind loads. These checks will typically govern the size of the top flange and may influence the cross-frame or diaphragm spacing, in regions of positive flexure. The basic philosophy behind these provisions is that no nominal yielding and no reliance on post-buckling resistance is permitted for main load-carrying members during construction. These checks are necessary since girders in the positive moment region have significantly less load-carrying capacity in their noncomposite condition, and many of the loading conditions which occur during construction are not considered in the evaluation of the bridge in its final, completed condition.

It should be noted that some of the loads which occur during construction, such as deck overhang loads and wind loads, induce flange lateral bending stresses. Most line girder analysis programs do not directly calculate flange lateral bending stresses. Line girder analysis programs use one-dimensional analysis models which cannot directly address the torsional loading effects which produce flange lateral bending. Some programs will allow the user to manually input flange lateral bending moments or stresses, calculated by the user outside the program; other programs may not have this ability. In many cases, it may be easiest to evaluate the effects of flange lateral bending on the design outside of a line girder analysis program using hand calculations, perhaps facilitated by programming these calculations in a spreadsheet. This allows the designer flexibility, particularly in addressing the various constructibility checks which must be performed. If it turns out that consideration of the effects of flange lateral bending, whether in a constructibility check, or in the supplemental checks of the bottom flange under final conditions, results in a controlling design condition, the designer may have to rerun the line girder analysis with resized girders to update major-axis bending stress calculations, deflection calculations, or other calculations.

Load combinations for the constructibility design checks include Strength I, Strength III, and a "Special" load combination for primary steel superstructure components specified in Article 3.4.2.1. Owners may have explicit or implicit policies which prescribe specific loads, load combinations, and other assumptions for constructibility checks which supplement or take the place of those presented in the AASHTO LRFD BDS.

In general, for Strength I, Article 3.4.2.1 prescribes various load factors, including a load factor of 1.5 to be applied to construction loads. Wind is not included in the Strength I load combination.

Strength III is for dead load in combination with wind load; often this limit state is used to represent a case of inactive construction, where the structural steel is erected and in place but no construction activity is occurring. This might model an overnight or weekend condition when construction is not occurring. For this load combination, dead load includes the self-weight of the structural steel and perhaps some various construction dead loads in place, depending on Owner-agency policy and what might be permitted in the Owner-agencies standard specifications or in the contract documents. Typically, this load combination includes no construction live load but does include a high wind load. Article 3.4.2.1 states that the load factor on the wind load for the Strength III load combination during construction is to be specified by the Owner-agency. If the Owner-agency does not provide guidance, the AASHTO *Guide Specifications for Wind Loads on Bridges During Construction* can be consulted. The load factor on any construction dead loads that are included in this load combination with the self-weight of the structural steel is not to be less than 1.25.

The "Special" load combination prescribed in Article 3.4.2.1 typically represents a case of active construction. For this load combination the dead load includes the self-weight of the structural steel, the dead load of the wet concrete deck (including consideration of the prescribed deck-pour sequence), and construction dead loads such as formwork, falsework, etc. The live load consists of construction live load, including consideration of both construction equipment and construction personnel using a load factor of 1.4 for DC (component) dead loads and any construction loads acting on the fully erected steelwork, including dynamic load effects (if applicable). This load combination typically includes a reduced wind load or no wind load, depending on Owner-agency policy.

Construction loads or loads that act on the structure only during construction loads include, but are not limited to, the weight of materials, removable forms, personnel, and equipment such as deck finishing machines or loads applied to the structure through falsework or other temporary supports. The weight of the wet concrete deck and any stay-in-place forms should be considered as DC loads.

Failure modes of concern during construction of flexural members in the routine steel I-girder bridges covered by this Guide include nominal yielding, local buckling, lateral torsional buckling, web bend buckling, and/or shear buckling. All constructibility design checks are to be made on the noncomposite steel girder and are typically critical only in regions of positive flexure.

The potential for uplift at bearings during construction should be considered but is generally not a concern for the routine steel I-girder bridges covered by this Guide, except possibly for multi-span continuous bridges with short end spans or other issues related to poor span balance. Also, the provisions of Article D6.5 need only be checked for routine rolled-beam bridges at bearings.

For additional information on the design checks for constructibility, consult the NSBA's Steel Bridge Design Handbook – Chapter 11: Design for Constructability. For design examples illustrating constructibility design computations, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge, and Sections 3.3 and 6.5.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. Also see the Discussion of Article 3.4.2 in this Guide for more detailed information on the required loads and load combinations to consider for the constructibility checks. The Reference Manual for NHI Course 130102, Engineering for Structural Stability in Bridge Construction provides further discussion of constructibility checks and the associated loads.

The reader is cautioned that these references have not yet been updated to reflect changes made in the 10^{th} Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10^{th} Edition of the AASHTO LRFD BDS.

6.10.3.2 Flexure

6.10.3.2.1 Discretely Braced Flanges in Compression

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article contains three equations to check the flexural resistance of the discretely braced top (compression) flange of the bare steel section in regions of positive flexure for the combined effects of either the DC_1 loads (for simple span bridges only) or the deck-placement sequence (for multi-span continuous bridges only) and the deck overhang loads, as applicable, during the construction of the concrete deck. f_{bu} is taken as the factored major-axis compressive stress in the top flange due to the moment acting on the noncomposite section at the section under consideration (see the discussion below). When investigating the deck-placement sequence (see the Discussion of Article 6.10.3.4.1 in this Guide for more information on the deck placement sequence), the moment used to calculate f_{bu} should be the maximum accumulated moment occurring on the noncomposite section only during the placement sequence. For the exterior (fascia) girders, f_ℓ is taken as the factored flange lateral bending stress in the top flange due to the deck overhang loads at the section under consideration (see the discussion below and also the Discussion of Article 6.10.3.4.1 in this Guide for more information on deck overhang loads); the flange lateral bending stress in this case will often be subject to amplification (see the Discussion of Article 6.10.1.6 in this Guide). Each of the three equations in this Article are applicable to the routine steel I-girder bridges covered by this Guide.

When checking these equations with the section in its noncomposite condition, the categorization of the web is to be based on the properties of the noncomposite section. See the Commentary for Article 6.10.6.2.3 and the Discussion of Article 6.10.6.2.3 in this Guide for further discussion on the definition and categorization of the compact web, noncompact web, and slender web sections mentioned below.

Eq. 6.10.3.2.1-1 is a yielding limit state check. This equation does not need to be checked for interior girders utilizing slender web sections (where f_ℓ due to the deck overhang brackets is equal to zero) as this equation will not control.

Eq. 6.10.3.2.1-2 checks that the member has sufficient strength with respect to flange local buckling (see the Discussion of Article 6.10.8.2.2 or A6.3.2 in this Guide, as applicable) and lateral-torsional buckling (see the Discussion of Article 6.10.8.2.3 or A6.3.3 in this Guide, as applicable). In computing the flange local buckling and lateral-torsional buckling resistances, the web load-shedding factor, R_b, (see the Discussion of Article 6.10.1.10.2 in this Guide) is taken equal to 1.0 since web bend-buckling is explicitly prevented during construction via Eq. 6.10.3.2.1-3 (see below). When lateral-torsional buckling controls, the stress, f_{bu}, used to check Eq. 6.10.3.2.1-2 is to be determined as the value of the flange compressive stress at the cross-section where $f_{bu}/R_bR_hF_{yc}$ is maximum in the unbraced length under consideration, including the end cross-sections, calculated without consideration of flange lateral bending. R_b is to be taken equal to 1.0, as discussed above. R_h is the hybrid factor (see the Discussion of Article 6.10.1.10.1 in this Guide) taken equal to 1.0 since the routine steel I-girder bridges covered by this Guide do not utilize hybrid-section members. The largest value of f_ℓ within the unbraced length is also used.(see the Discussion of Article 6.10.1.6 in this Guide). For sections with compact or noncompact webs, the lateral-torsional buckling resistance for use in Eq. 6.10.3.2.1-2 may optionally be computed according to the provisions of Article A6.3.3 (to include the beneficial contribution of the St. Venant torsional stiffness, J), which is recommended in particular for routine rolled-beam bridges with larger unbraced lengths during construction (see the Discussion of Articles 6.10.6.2.3 and A6.3.3 in this Guide). The resulting lateral-torsional buckling resistance, M_{nc}, in this case is divided by S_{xc} as defined in this Article to express the resistance in terms of stress for direct application in Eq. 6.10.3.2.1-2. In some cases, the calculated resistance may exceed F_{yc}. However, Eq. 6.10.3.2.1-1 will control in such cases, or perhaps Eq. 6.10.1.8-1 if there are holes in the tension flange at the section under consideration (see the Discussion of Article 6.10.1.8 in this Guide), thus ensuring that the combined factored stress in the flange will not exceed F_{yc} during construction. When flange local buckling controls, f_{bu} and f_ℓ used to check Eq. 6.10.3.2.1-2 may be determined as the corresponding values at the section under consideration (see the Discussion of Article 6.10.1.6 in this Guide).

Eq. 6.10.3.2.1-3 checks that theoretical web bend-buckling (Article 6.10.1.9.1) will not occur during construction. This equation does not need to be checked for sections with compact or noncompact webs (e.g., sections in routine rolled-beam bridges). Options to consider should the web bend-buckling stress be exceeded are discussed in the last paragraph of the Commentary for this Article.

For routine multi-span continuous bridges, these three equations should also be used to check the discretely braced bottom (compression) flange of the bare steel section in regions of negative flexure; however, the checks given by the first two equations typically do not control.

As explained in the Discussion of Article 6.10.3.1 in this Guide, designers are reminded that most commercial steel bridge line girder analysis and design software packages are not capable of calculating all of the design stresses associated with the constructibility checks. Generally, the constructibility checks are performed outside of the line girder analysis and design program, using hand calculations, perhaps facilitated by programming these calculations in a spreadsheet.

6.10.3.2.2 Discretely Braced Flanges in Tension

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article contains an equation to check the flexural resistance of the discretely braced bottom (tension) flange of the bare steel section in regions of positive flexure for the combined effects of either the DC_1 loads (for simple span bridges only) or the deck-placement sequence (for multi-span continuous bridges only) and the deck overhang loads, as applicable, during the construction of the concrete deck. f_{bu} is taken as the factored major-axis compressive stress in the bottom flange due to the moment acting on the noncomposite section at the section under consideration. When investigating the deck-placement sequence (see the Discussion of Article 6.10.3.4.1 in this Guide for more information on the deck-placement sequence), the moment used to calculate f_{bu} should be the maximum accumulated moment occurring on the noncomposite section only during the placement sequence. For the exterior (fascia) girders, f_ℓ is taken as the factored flange lateral bending stress in the bottom flange due to wind loads and the deck overhang loads at the section under consideration (see the Discussion of Article 6.10.3.4.1 in this Guide for more information on deck overhang loads); the flange lateral bending stress in this case is not subject to amplification (see the Discussion of Article 6.10.1.6 in this Guide) since the flange is in tension. The equation in this Article, which is a yielding limit state check, is applicable to the routine steel I-girder bridges covered by this Guide but typically does not control.

For routine multi-span continuous bridges, this equation should also be checked for the discretely braced top (tension) flange of the bare steel section in regions of negative flexure; however, this check typically does not control.

As explained in the Discussion of Article 6.10.3.1 in this Guide, designers are reminded that most commercial steel bridge line girder analysis and design software packages are not capable of calculating all of the design stresses associated with the constructibility checks. Generally, the constructibility checks are performed outside of the line girder analysis and design program, using hand calculations, perhaps facilitated by programming these calculations in a spreadsheet.

6.10.3.2.3 Continuously Braced Flanges in Tension or Compression

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article contains an equation to check the flexural resistance of top flange of the beam or girder subject to tension or compression for any factored construction loads that may be applied after the deck has hardened and is continuously braced. A continuously braced flange is defined as a flange that is encased in concrete or anchored to the concrete deck by shear connectors satisfying the provisions of Article 6.10.10. In some multi-span continuous routine steel I-girder bridges, Owner-agency policy may prescribe that shear connectors not be provided in regions of negative flexure; in those regions the provisions of this Article would not apply.

Flange lateral bending need not be considered in a continuously braced flange. The lateral resistance of the concrete deck is generally adequate to compensate for the neglect of any initial lateral bending stresses in the steel prior to placement of the deck and any additional lateral bending stresses induced after the deck has been placed. Flange local buckling and lateral-torsional buckling also need not be considered when a continuously braced flange is subject to compression.

As explained in the Discussion of Article 6.10.3.1 in this Guide, designers are reminded that most commercial steel bridge line girder analysis and design software packages are not capable of calculating all of the design stresses associated with the constructibility checks. Generally, the constructibility checks are performed outside of the line girder analysis and design program, using hand calculations, perhaps facilitated by programming these calculations in a spreadsheet.

6.10.3.2.4 Concrete Deck

Determination of applicability, *Simple Span Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Applicable.

Discussion:

This Article requires that a minimum area of longitudinal reinforcement equal to at least one-percent of the concrete deck cross-sectional area is provided in multi-span continuous bridges wherever the factored tensile stress in the deck (see the Discussion of Article 6.10.1.1.1d in this Guide) during the deck-placement sequence exceeds $0.9f_r$, where f_r is the modulus rupture of the concrete taken equal to $0.24\sqrt{f_c'}$ for the routine steel I-girder bridges covered by this Guide. See the Discussion of Article 6.10.1.7 in this Guide for further discussion of this provision.

Simple Span Bridges:

The minimum one-percent longitudinal reinforcement in the concrete deck specified in this Article is not required for crack control in routine simple span I-girder bridges because the deck is in compression; hence, this Article is not applicable for the design of simple span routine steel I-girder bridges. Temperature and shrinkage reinforcement is still required in the decks of these types of bridges, however.

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

The provisions of this Article are applicable to the routine steel multi-span continuous I-girder bridges covered by this Guide.

6.10.3.3 Shear

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article provides an equation to check the web for the sum of the factored permanent loads and factored construction loads applied to the noncomposite section during construction. The equation in this Article is applicable to the routine steel I-girder bridges covered by this Guide.

The factored shear resistance for this check is limited to the shear-yielding or shear-buckling resistance (see the Discussion of Article 6.10.9 in this Guide). The use of post-buckling tension-field action is not permitted to resist construction loads. Use of tension-field action is permitted at the strength limit state after the deck has hardened or is made composite (if the section along the entire web panel is proportioned according to the requirements for developing tension-field action discussed in Article 6.10.9).

The shear in the end panels of stiffened webs is already limited to either the shear-yielding or shear-buckling resistance, as is the shear in unstiffened webs. Therefore, this requirement typically does not need to be checked for unstiffened webs or for the end panels of stiffened webs.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Since the shear in the web is typically not affected by lateral and torsional loading effects such as wind load and overhang bracket loading, these types of programs may be able to directly perform the shear constructibility checks specified by this Article. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.3.4 Deck Placement

6.10.3.4.1 General

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Applicable.

Discussion:

The provisions of this Article are applicable to the multi-span continuous routine steel I-girder bridges covered by this Guide, while some of the provisions specifically related to the effects of sequential deck placement on the behavior of multi-span continuous bridges may not be directly applicable to simple-span routine steel I-girder bridges.

Consideration of Deck Placement Sequence

A sequential deck placement analysis need not be performed for simple-span bridges as it will not control over the case assuming the entire deck is placed at once. For multi-span continuous bridges, on the other hand, the effects of the deck placement sequence must be considered.

The provisions require that sections in positive flexure in multi-span continuous bridges that are composite in the final condition, but noncomposite during construction, be investigated during the various stages of the deck placement for a specified deck placement sequence shown in the contract documents. This Article refers to Article 6.10.3.2 for the checking of the bare steel girder in regions of positive flexure only for the effects of the placement sequence, and for the exterior girder, the effect of the deck overhang loads (see the Discussion of Article 6.10.3.2 in this Guide). The bare steel girder should be checked for the maximum accumulated moment acting on the noncomposite section only during the placement sequence.

Changes in load, stiffness and bracing during the various stages of the deck placement in multi-span continuous bridges must be considered. During deck placement, the actual composite stiffness depends on the amount of time that the concrete has had to cure before the next portion is cast, but such refinements are usually not considered in the analysis. Unless a retarder is used, concrete usually obtains composite action in a matter of hours after placement. Thus, the full composite stiffness is often used for the previously placed concrete.

Common practice when casts include both positive and negative moment regions is to cast the slab in the positive moment regions first, and then cast the slab in the negative bending region over the support to minimize cracking at the top of the slab. However, when concrete is cast in a span adjacent to a span that already has a hardened deck, induced negative moments in the adjacent spans will cause tensile stresses in the cured concrete that may result in transverse deck cracking. Provision of the minimum required one-percent longitudinal reinforcement in the deck at these sections can help control the cracking (see the Discussion of Articles 6.10.3.2.4 and 6.10.1.7 in this Guide). In a long cast, e.g. extending from one end of the bridge over an interior support into an adjacent span, it is possible that the concrete in the negative moment region over the support will harden and be subject to tensile stresses during the remainder of the cast, which may result in early age cracking of the deck. A retarder admixture may be required in the casts over the piers to reduce the potential for early age deck cracking. In such cases, the end span must still be checked for the critical instantaneous unbalanced case where wet concrete exists over the entire end span, with no concrete yet on the remaining spans.

Temporary dead load deflections during the sequential deck placement can also be different from the final noncomposite dead load deflections. If the differences are deemed significant, this should be considered when establishing camber requirements.

When computing deflections considering staged deck placement, the stiffness of previously cast portions of the concrete deck can potentially be based on a modular ratio closer to the short-term modular ratio, *n*, since the concrete does not have enough time to creep significantly between casts. Considering this effect increases the complexity of the analysis and should only be undertaken when required by Owner-agency policy or when the designer deems that the nature of the deflections truly warrants this level of refinement. At least one State DOT has found the use of a concrete modulus of elasticity equal to 70 percent of the modulus of elasticity at 28 days (which results in a modular ratio of approximately 1.4*n* for transforming the section) to be appropriate in computing the stiffness.

The NSBA's LRFD Simon line-girder analysis and design program, along with other available commercial software, generally have options available to perform a sequential deck placement analysis as described above. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

Investigation of Uplift During Deck Placement

Potential uplift at the bearings should also be investigated during the deck placement, although uplift is generally not a concern for the routine steel I-girder bridges covered by this Guide, except possibly for multi-span continuous bridges with short end spans or other issues related to poor span balance. In cases where the potential for uplift may be a concern, it is suggested in the absence of other Owner-agency guidance that a load factor of 1.0 be applied to all downward support reactions caused by component dead loads contributing to uplift, and a load factor of 0.9 be applied to all upward support reactions caused by component dead loads resisting uplift. This investigation of potential for uplift can usually focus on critical construction stages and can typically be accomplished using hand calculated modifications of the bearing reactions reported by the line girder analysis program.

Torsion Caused by Deck Overhang Brackets

The effect of the torsion due to the forces from deck overhang brackets acting on the exterior (fascia) girders must also be considered for both the routine multi-span continuous and simple-span I-girder bridges covered by this Guide. The deck overhang weight is resisted by the brackets. If the bracket is assumed to extend near the edge of the deck overhang, it can be assumed that half of the deck overhang weight is placed on the fascia girder and half is placed on the overhang brackets. One-half of the deck haunch weights can also conservatively be included in the total overhang weight. Besides the weight of the deck, typical loads that may act on the overhang only during construction include the overhang deck forms, the screed rail, the railing, the walkway, the overhang brackets and the finishing machine. Designers should consider talking with local contractors to obtain reasonable estimates of the load values. The vertical load on the overhang can then be resolved into lateral forces, *F*, acting on the flanges. The lateral forces are dependent on the assumed angle that the bracket makes with the web; overhang brackets bearing near the bottom flange is the preferred configuration. The deck overhang brackets may bear on the girder web only if means such as blocking, bracing, or other stiffening are provided so that the web is not subject to excessive out-of-plane bending stresses or otherwise damaged, and so that the resulting deflections of the overhang falsework do not adversely affect proper placement and screeding of the bridge deck concrete. The lateral forces on the top flange and web increase when the bracket bears directly on the web.

The approximate equations given in the Commentary for this Article may be used to estimate the flange lateral bending moments at the cross-frames or diaphragms due to the lateral flange forces. Eq. C6.10.3.4.1-1 applies if a statically equivalent uniformly distributed bracket force, F_ℓ, is assumed; bracket dead loads are typically assumed applied uniformly. Eq. C6.10.3.4.1-2 may be used if the finishing machine load is assumed applied as a single concentrated load.

For a discretely braced compression flange, the lateral bending stress due to the overhang bracket load will often be subject to amplification (see the Discussion of Article 6.10.1.6 in this Guide). The first-order flange lateral bending stress, $f_{\ell 1}$, is determined by dividing the lateral bending moment by the lateral section modulus of the flange (i.e., $t_f b_f^2/6$). The amplified lateral bending stress is subject to the specified limit on lateral bending stress of $0.6F_{yf}$ (Article 6.10.1.6). Major-axis bending moments due to the overhang construction loads (e.g., formwork, walkways, and finishing machine loads) are typically not considered because these loads are usually much smaller in magnitude relative to other design loads on the bridge. Also, overhang construction loads represent a temporary loading condition that is not present in the finished structure. Lateral bending moments due to these loads are usually much more critical. The magnitude and application of the overhang loads assumed in the design should also be shown on the contract documents.

In some cases, the flange lateral bending effects resulting from overhang bracket loads can be significant, particularly when shallow depth girders are being used. In such cases, reducing the width of the permanent bridge deck overhang or reducing the width or eccentricity of the temporary overhang brackets, construction access, and construction appurtenances (such as the deck screed rail) may be beneficial.

Finally, it should be noted that the overhang brackets often support the rails on which the deck screed runs when finishing the deck; torsional deformation (twisting) of the girders due to the eccentric loading applied by the overhang brackets contributes to vertical deflections of the screed rail. These deflections affect the profile to which the deck is finished, and excessive deflection can result in improper deck thickness. In such situations, additional temporary bracing between the exterior and first interior girder is often used to reduce the torsional deformation of the exterior girder. The evaluation of this effect is typically the responsibility of the Contractor's specialty engineer and is usually addressed in the design of the overhang falsework system, but occasionally Owner-agencies require the designer to evaluate this effect; see local Owner-agency policy. Software such as the TAEG program is sometimes used to automate these calculations (https://kart.ksdot.org/).

Analysis of Deck Placement during Phased Construction or Bridge Widening

The analysis of the effects of deck placement during phased construction or bridge widening warrants special discussion. In these situations, there will be instances where some or all of the weight of a wet concrete deck placement will represent loading on a portion of a routine steel I-girder bridge which already has a composite concrete deck in place. In those cases, the loading on the composite portion of the bridge should be treated as a composite dead load, applied to the composite section for calculating girder and deck stresses.

For example, consider a bridge built in two phases. Assume Phase 1 has five girders (numbered Girder 1 through Girder 5), spaced 9'-0" center to center, and that at Girder 5 the deck overhang in the temporary Phase 1 condition is 2'-0". Later Phase 2 is built. Assume Phase 2 also has five girders (numbered Girder 6 through Girder 10), spaced 9'-0" center to center, and that at Girder 6 the deck overhang is 2'-0". Assume a 5'-0" closure pour will be placed, connecting Phase 1 to Phase 2, after the five Phase 2 girders are built and their deck is cast. In this case, the weight of the wet concrete deck for the Phase 1 deck construction represents a noncomposite dead load on Girders 1 through 5, and the weight of wet concrete deck for the Phase 2 deck construction represents a noncomposite dead load on Girders 6 through 10. The weight of the wet concrete deck of the closure pour including any associated deck forms and any cross-frames added between the two phases, on the other hand, represent composite dead loads on the two previously completed portions of the bridge. Since each of those previous phases of construction are fairly wide, and the closure pour load is concentrated between them, the majority of the associated closure pour loads would be carried by Girders 5 and 6, with some of the loads carried by Girders 4 and 7 and possibly some by Girders 3 and 8, etc.

The actual load distribution, and the application of loading to composite or noncomposite sections, would need to be evaluated on a case-by-case basis, depending on the geometry and structural configuration of the bridge during each phase of construction. Consultation with senior bridge engineers experienced in the design of steel girder bridges built using phased construction methods is strongly encouraged when determining how to approach the analysis and design of these types of structures.

Further discussion of analysis and design issues associated with phased construction and widening of steel girder bridges can be found in Section 3.17 of AASHTO-NSBA Steel Bridge Collaboration's Guideline G13.1-2019 Guidelines for Steel Girder Bridge Analysis, and in Sections 1.6.2 and 2.2.2 of AASHTO-NSBA Steel Bridge Collaboration's Guideline G12.1-2020 Guidelines to Design for Constructability and Fabrication.

Deck Placement Sequence and Overhang Bracket Design Example References

For design examples illustrating the sequential deck placement and deck overhang calculations, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.10.3.4.2 Global Displacement Amplification in Narrow I-Girder Bridge Units

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides specific guidelines for checking the global stability of spans of slender unsupported straight or horizontally curved multiple I-girder bridge units interconnected by cross-frames or diaphragms when in their noncomposite condition during the deck placement. The provisions apply only to spans of I-girder bridge units with three or fewer girders (e.g., very narrow bridges, or partial-width stages of bridges that may occur during a phased construction or bridge widening scenario). Therefore, these provisions are not applicable to the routine steel I-girder bridges covered by this Guide since these bridges consist of four or more girders.

As noted in the "definition of a routine steel I-girder bridge" for the purposes of this Guide, all phases of phased construction projects or bridge widenings should also meet the definition of a routine steel I-girder bridge (including the requirements related to framing plan geometry and minimum numbers of girders, etc.); if a widening or a given phase of construction meets the definition of a routine steel I-girder bridge, then the provisions of this Article should not be applicable. If, however, the widening or a given phase of construction is narrow and has three or fewer girders, the provisions of this Article should be considered, and global stability should be investigated.

6.10.3.5 Dead Load Deflections

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article is considered self-explanatory and applicable to the routine steel I-girder bridges covered by this Guide. The Article simply refers to the provisions of Article 6.7.2 for further information on establishing the required vertical camber to compensate for the computed dead load deflections (see the Discussion of Article 6.7.2 in this Guide).

6.10.4 Service Limit State

6.10.4.1 Elastic Deformations

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article points to optional span-to-depth ratios and live-load deflection requirements in Article 2.5.2.6 to control elastic deformations so that the bridge will perform satisfactorily over its regular service life. This Article is applicable to the routine steel I-girder bridges covered by this Guide. Most Owners choose to enforce a live-load deflection requirement at the service limit state; consult the applicable Owner-agency standard. See the Discussion of Article 2.5.2.6.2 in this Guide for a full explanation of these provisions, their applicability, and how to implement them.

6.10.4.2 Permanent Deformations

6.10.4.2.1 General

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Applicable.

Discussion:

The provisions of Article 6.10.4.2 are intended to prevent objectionable permanent deformations of the steel beams or girders, caused by localized yielding and potential web bend-buckling under expected severe traffic loadings, which might impair rideability. The requirements in this Article must be satisfied under the Service II load combination (Table 3.4.1-1), which can be characterized in a simplified manner as being approximately halfway between the load combination used for Service I and Strength I. The load level for this limit state can reasonably be expected to be exceeded less than once every six months on average. The live load used for Service II load combination is HL-93 placed in one or more lanes. Although not covered in this Article, slip in bolted slip-critical connections is also to be checked at the service limit state under the Service II load combination. The provisions of Article 6.10.4.2 are applicable to the routine steel multi-span bridges, and partially applicable to the routine simple-span bridges, covered by this Guide. The checks in this Article can often control the size of the bottom flange of these bridges in regions of positive flexure, and the web thickness in regions of negative flexure in multi-span continuous bridges.

The provisions of Article 6.10.4.2.1 are intended to determine whether the concrete deck may be considered effective in tension for loads applied to the composite section in regions of negative flexure for the service limit state checks specified in Article 6.10.4.2.2 in routine multi-span continuous bridges. It is recommended that the deck be considered effective in tension if the following three requirements are satisfied:

1. shear connectors are provided along the entire length of the member;
2. the minimum one-percent longitudinal reinforcement in the concrete deck is provided in accordance with Article 6.10.1.7 (see the Discussion of Article 6.10.1.7 in this Guide); and
3. the maximum longitudinal tensile stress in the concrete deck under the Service II load combination is less than $2f_r$, where f_r is the modulus rupture of the concrete taken equal to $0.24\sqrt{f_c^{'}}$ for the routine steel I-girder bridges covered by this Guide.

Otherwise, the steel section alone or the steel section plus the longitudinal reinforcement within the effective width of the concrete deck is to be used (if shear connectors are provided) depending on the preferences of the Owner-agency.

Note that the check to determine whether the concrete deck may be considered effective in tension for the service limit state checks is not applicable for a simple span because the concrete deck is in compression over the entire length of the span.

6.10.4.2.2 Flexure

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

The provisions of this Article are partially applicable to the routine steel I-girder bridges covered by this Guide as described in the following.

This Article provides flange stress limits to indirectly control permanent deformations in steel flexural members at the service limit state. Only Eqs. 6.10.4.2.2-1 and 6.10.4.2.2-2 are applicable to the routine steel I-girder bridges covered by this Guide because shear studs are assumed provided along the full length (or portions) of the beam or girder. Should shear connectors be discontinued in regions of negative flexure in a multi-span continuous bridge, which is permitted but not recommended, Eqs. 6.10.4.2.2-1 and 6.10.4.2.2-2 are to be applied, as applicable, and not Eq. 6.10.4.2.2-3. For hybrid sections in which the Service II flexural stress, f_f, in both flanges does not exceed the specified minimum yield strength of the web, the hybrid factor, R_h, is to be taken equal to 1.0 in Eqs. 6.10.4.2.2-1, 6.10.4.2.2-2, and 6.10.4.2.2-3. However, the routine steel I-girder bridges covered by this Guide do not utilize hybrid-section members. Flange lateral bending stresses at the service limit state are not a concern for the routine I-girder bridges covered by this Guide as the effects of torsion due to deck overhang bracket loads and wind loads are not considered at the service limit state. Eqs. 6.10.4.2.2-1 and 6.10.4.2.2-2 do not control and need not be checked for sections in negative flexure in multi-span continuous bridges designed according to the provisions of Article 6.10.8 at the strength limit state (i.e., bridges utilizing slender web sections or compact or noncompact web sections treated as slender web sections in regions of negative flexure), and for sections in positive flexure treated as noncompact sections at the strength limit state (see the Discussion of Article 6.10.7 in this Guide).

A web bend buckling check is also specified in this Article at the service limit state to control bending deformations and transverse displacements in the compression zone of the web (Eq. 6.10.2.2.2-4). Regions in negative flexure in multi-span continuous bridges utilizing slender web sections are particularly susceptible to web bend buckling in composite girders at the service limit state, especially when the concrete deck is considered effective in tension as permitted for composite sections in Article 6.10.4.2.1 when certain specified conditions are satisfied. When the concrete deck is considered effective in tension in regions of negative flexure, more than half of the web is likely to be in compression increasing the susceptibility of the web to bend buckling. As a result, the check in this case may often end up governing the web thickness of the girder in these regions when the concrete is assumed effective. Should the necessary requirements be met such that the concrete may be considered effective in tension as permitted in Article 6.10.4.2.1, it is required that the elastic depth of the web in compression, D_c, be computed according to the provisions of Article D6.3.1 for the web bend-buckling check given by Eq. 6.10.4.2.2-4 (see the Discussion of Article D6.3.1 in this Guide). The provisions of Article D6.3.1 account for the fact that part of the load is applied to the noncomposite section, and thus provide a more accurate location of the neutral axis based on the total factored stresses. For composite sections in regions of positive flexure at the service limit state, web bend-buckling will typically not control because D_c is small in these regions in the routine steel I-girder bridges covered by this Guide, and so the check is waived. Options to consider should Eq. 6.10.4.2.2-4 be violated include providing a larger compression flange or a smaller tension flange to reduce D_c or providing a thicker web. Because an explicit web bend-buckling check is specified at the service limit state, the web load-shedding factor, R_b, (see the Discussion of Article 6.10.1.10.2 in this Guide) is not included in the equations of this Article.

The web bend-buckling check given by Eq. 6.10.4.2.2-4 need not be checked for simple-span routine steel I-girder bridges covered by this Guide since the beam or girder is subject to positive flexure only and in those cases, D_c is small and thus the potential for web bend-buckling is negligible.

The web bend-buckling check given by Eq. 6.10.4.2.2-4 also need not be checked for routine multi-span continuous rolled beam bridges covered by this Guide. In regions of negative flexure, theoretical web bend-buckling will not occur for elastic stress levels, computed according to beam theory, at or below F_{yc} for a compact or noncompact web section. In regions of positive flexure, D_c is small and thus the potential for web bend-buckling is negligible.

The definition of a routine steel I-girder bridge specifically excludes the use of moment redistribution methods and so the optional provisions of Appendix B6 mentioned in the Commentary for this Article are considered not applicable to the routine steel I-girder bridges covered by this Guide.

The provision in this Article related to compact composite sections in positive flexure utilized in shored construction is not applicable; shored construction is not recommended for routine steel I-girder bridges.

For further information on service limit state design, consult Section 6.5.4 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples illustrating service limit state design checks to limit permanent deformations, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.5 Fatigue and Fracture Limit State

6.10.5.1 Fatigue

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article refers to the provisions of Article 6.6 for checking fatigue of details using the fatigue live load (see the Discussion of Articles 6.6 and 3.6.1.4 in this Guide) and the appropriate Fatigue load combination (Table 3.4.1-1), and to the provisions of Article 6.10.10.2 or 6.10.10.3 (as applicable) for determining the nominal fatigue resistance of shear connectors (see the Discussion of Articles 6.10.10.2 and 6.10.10.3 in this Guide). These provisions are applicable to the routine steel I-girder bridges covered by this Guide.

The provision for checking the fatigue stress range due to major-axis bending plus flange lateral bending in horizontally curved I-girder bridges is not applicable to the routine steel I-girder bridges covered by this Guide.

For further information on fatigue limit state design of shear connectors, consult the Discussion of Article 6.10.10.1.2 in this Guide and Section 6.6.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS. For design examples illustrating fatigue limit state design of shear connectors, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.5.2 Fracture

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article refers to Article 6.6.2.1 for provisions related to the fracture toughness requirements (i.e., Charpy V-Notch toughness requirements) specified in the contract documents. This Article is applicable to the routine steel I-girder bridges covered by this Guide. For further explanation, see the Discussion of Article 6.6.2.1 in this Guide.

6.10.5.3 Special Fatigue Requirement for Webs

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The provision in this Article is intended to control significant elastic flexing of the web under repeated live loads by limiting the factored shear for this check, V_u, to the shear buckling resistance of the web, V_{cr} (see the Discussion of Article 6.10.9.1 in this Guide). V_u for this check is taken equal to the unfactored permanent load shear plus the factored Fatigue I shear due to the fatigue live load plus the applicable dynamic load allowance (see the Discussion of Articles 3.6.1.4 and 3.6.2.1 in this Guide); that is, this check is to be done for the heaviest truck expected to cross the bridge in 75 years. This provision is applicable to the routine steel I-girder bridges covered by this Guide.

If post-buckling tension-field action were permitted under this load condition, the principal tensile stress range acting along the buckle would result in significant out-of-plane flexing of the web under repeated live loads, which would be undesirable. By limiting the factored shear under this load condition to the shear buckling resistance, V_{cr}, the member is assumed to be able to sustain an infinite number of smaller loadings without fatigue cracking due to this effect.

Eq. 6.10.5.3-1 need not be checked for unstiffened webs or end panels of stiffened webs because the shear in these cases is already limited to V_{cr} at the strength limit state (see the Discussion of Article 6.10.9 in this Guide).

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program’s calculations prior to initial use.

6.10.6 Strength Limit State

6.10.6.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article refers to the applicable Strength load combinations given in Table 3.4.1-1, which are utilized in the design checks at the strength limit state for the routine steel I-girder bridges covered by this Guide.

Note that the Commentary for this Article contains the following useful information:

- Explanation of why flexural resistances at the strength limit state are expressed in terms of stress or moment in different parts of the specification;
- Guidance on correctly interpreting and applying the results from refined analyses at the strength limit state (although refined methods of analysis are not necessary or recommended for the design of routine steel I-girder bridges);
- Discussion about continuously braced flanges; and
- Discussion about the level of axial force at which a member can be solely designed as a flexural member.

6.10.6.2 Flexure

6.10.6.2.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article refers to the provisions Article 6.10.1.8 if there are holes in the tension flange of an I-section flexural member; e.g., at a bolted splice (see the Discussion of Article 6.10.1.8 in this Guide). These provisions are applicable to the routine steel I-girder bridges covered by this Guide.

For further information on strength limit state design for flexure, consult Section 6.5.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS. For design examples illustrating strength limit state design flexure checks, consult the as well as NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.6.2.2 Composite Sections in Positive Flexure

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article defines the requirements for a composite I-section in regions of positive flexure to qualify as a compact or a noncompact section at the strength limit state, and where the provisions to design each type of section are located in Article 6.10.7 (see the Discussion of Article 6.10.7 in this Guide). The provisions of this Article are partially applicable to the routine steel I-girder bridges covered by this Guide as described further below.

Most composite sections in regions of positive flexure in routine steel I-girder bridges will qualify as compact sections; that is, these bridges are straight, the specified minimum yield strength of the flanges does not exceed 70 ksi, web longitudinal stiffeners are not required, and sections will satisfy the web-slenderness limit given by Eq. 6.10.2.2.2-1, where D_{cp} is the depth of the web in compression at the plastic moment, M_p (see the Discussion of Articles D6.1 and D6.3.2 in this Guide). For composite sections in positive flexure in routine steel I-girder bridges, the plastic neutral axis (*PNA*) will almost always be located either in the top flange or in the concrete deck. In either case, D_{cp} will equal zero and Eq. 6.10.2.2.2-1 may be considered automatically satisfied.

For compact sections in regions of positive flexure, the nominal flexural resistance is permitted to exceed the moment at first yield provided there are no holes in the tension flange at the section under consideration, but for noncompact sections and for compact sections with holes in the tension flange it is not. The moment at first yield, M_y, is defined as the moment at which an outer fiber first attains the yield stress (see the Discussion of Article D6.2.2 in this Guide). The nominal flexural resistance is not permitted to exceed the plastic moment, M_p, for both noncompact and compact sections. M_p is defined as the resisting moment of a fully yielded cross-section (see the Discussion of Article D6.1 in this Guide). Another primary difference is that for noncompact sections, the nominal flexural resistance is expressed in terms of the elastically computed flange stress. However, for compact sections, the nominal flexural resistance is expressed in terms of moment. The reasons for this are discussed further in the Commentary for Article 6.10.6.1. Sections that qualify as compact sections without holes in the tension flange may conservatively be treated as noncompact sections, if desired. For compact sections, the strength limit state criteria are unlikely to control the size of the bottom flange (or the size of the rolled-beam section) in regions of positive flexure; service or fatigue limit state criteria are more likely to govern. For a noncompact section, strength limit state criteria are more likely to control the size of the bottom flange (or the size of the rolled-beam section).

This Article also refers to the ductility requirement given in Article 6.10.7.3 to provide a ductile mode of failure, which must be checked for both compact and noncompact sections (see the Discussion of Article 6.10.7.3 in this Guide).

The language at the beginning of this Article relative to a horizontally curved bridge or a kinked (chorded) continuous bridge is not applicable to the routine steel I-girder bridges covered by this Guide.

For further information on strength limit state design for flexure, consult Section 6.5.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples illustrating strength limit state design flexure checks, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.6.2.3 Composite Sections in Negative Flexure and Noncomposite Sections

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article defines the requirements for a composite I-section in regions of negative flexure and a noncomposite I-section in regions of positive or negative flexure to qualify as a slender web section or a nonslender web section, and the provisions that may be used to define each type of section at the strength limit state. These provisions are partially applicable to the routine steel I-girder bridges covered by this Guide as described further below. These criteria, when applied at the strength limit state, typically control the size of the top and bottom flange (or the size of the rolled-beam section) and the cross-frame or diaphragm spacing in regions of negative flexure in the routine steel multi-span continuous I-girder bridges covered by this Guide. These criteria can also control the size of the noncomposite section and the cross-frame or diaphragm spacing in regions of positive flexure during construction.

A slender web section contains a web with a slenderness at or above the limit at which the theoretical elastic bend-buckling stress of the web in flexure (see the Discussion of Article 6.10.1.9.1 in this Guide) is reached prior to reaching the yield strength of the compression flange,

F_{yc}. Otherwise, the section is a nonslender web section and is not subject to web bend-bucking prior to reaching its limit of flexural resistance. Sections with slender webs rely on significant web post bend-buckling resistance at the strength limit state. The limiting web slenderness ratio delineating a slender web section from a nonslender web section is called λ_{rw} and is given by Eq. 6.10.6.2.3-3 (see also Table 6.10.1.10.2-2). λ_{rw} is also referred to as the noncompact web slenderness limit. The web slenderness based on the depth of the web in compression in the elastic range, D_c (see the Discussion of Article D6.3.1 in this Guide), is checked against this limiting ratio (Eq. 6.10.6.2.3-1). For sections with a specified minimum yield strength of the compression flange, F_{yc}, equal to 50 ksi, the upper and lower limits of λ_{rw} are 111 and 137. Sections with a web slenderness exceeding λ_{rw} are termed slender web sections.

In the routine steel multi-span continuous I-girder bridges covered by this Guide, nonslender web sections in regions of negative flexure that also satisfy the provisions of Eq. 6.10.6.2.3-2, and do not have holes in the tension flange at the section under consideration, may optionally be designed at the strength limit state according to the provisions of Appendix A6 (see the Discussion of Appendix A6 in this Guide); otherwise, the section is treated as a slender web section and the provisions of Article 6.10.8 must be used to design the section (see the Discussion of Article 6.10.8 in this Guide). The provisions of Article 6.10.8 assume that the section is a slender web section regardless of whether it is or not. As such, the nominal flexural resistance of the section computed according to these provisions is not permitted to exceed the moment at first yield, and as a result, the resistance equations in Article 6.10.8 are expressed in terms of stress for the reasons discussed in the Commentary for Article 6.10.6.1. The nominal flexural resistance of nonslender web sections designed according to the provisions of Appendix A6 is permitted to exceed the moment at first yield. Hence, the resistance equations in Appendix A6 are expressed in terms of bending moment, again for the reasons discussed in the Commentary for Article 6.10.6.1. For noncomposite nonslender web sections during construction, the lateral-torsional buckling resistance for use in Eq. 6.10.3.2.1-2 may optionally be computed according to the provisions of Article A6.3.3 (to include the beneficial contribution of the St. Venant torsional stiffness, J).

Nonslender web sections are further categorized as either compact web sections or noncompact web sections in Appendix A6. A compact web section is one that satisfies the web slenderness limit based on the depth of the web in compression at the plastic moment, D_{cp} (see the Discussion of Article D6.3.2 in this Guide) given by Eq. A6.2.1-1. For sections with F_{yc} equal to 50 ksi, the limiting web slenderness from Eq. A6.2.1-1 based on D_{cp} is 91 for a shape factor, M_p/M_y, of 1.12 and 64 for a shape factor of 1.30. These limits would generally be satisfied by rolled shapes or plate girders with proportions similar to those of a rolled shape, which would typically be used in a shorter-span bridge (i.e., spans of about 120 ft or less). Sections with compact webs and I_{yc}/I_{yt} greater than or equal to 0.3 (Eq. 6.10.6.2.3-2) are able to develop their full plastic moment capacity, M_p, at the strength limit state provided that other steel grade, ductility, flange slenderness and/or lateral bracing requirements are satisfied. Note that I_{yc} and I_{yt} in Eq. 6.10.6.2.3-2 are to be computed about the vertical axis in the plane of the web (or about the strong axis of each flange).

Sections exceeding the web slenderness limit given by Eq. A6.2.1-1, but with a web slenderness based on the elastic depth of the web in compression, D_c (see the Discussion of Article D6.3.1 in this Guide), less than or equal to λ_{rw}, are termed noncompact web sections, which have a nominal flexural resistance at the strength limit state that linearly transitions from M_p to the moment at first yield, M_y (see the

Discussions of Articles D6.1 and D6.2 in this Guide) as a function of the web slenderness. The majority of routine steel-bridge plate-girder sections utilize either slender web sections or noncompact web sections that approach λ_{rw}.

The definition of a routine steel I-girder bridge specifically excludes the use of moment redistribution methods and so the optional provisions of Appendix B6 mentioned in this Article are considered not applicable to the routine steel I-girder bridges covered by this Guide.

The language at the beginning of this Article relative to a horizontally curved bridge or a kinked (chorded) continuous bridge is not applicable to the routine steel I-girder bridges covered by this Guide.

Some specific guidance on the applicability of the provisions in this Article to various types of routine steel I-girder bridges is provided below.

The provisions of this Article are not applicable at the strength limit state for the routine simple-span bridges covered by this Guide, including the provisions of the optional Appendix A6, because simple-span bridges are subject to positive flexure only. However, the provisions of Article A6.3.3 may optionally be applicable for the noncomposite section in a simple span bridge during construction if the noncomposite section qualifies as a compact web or noncompact web section.

Sections in regions of negative flexure in the routine multi-span continuous rolled beam bridges covered by this Guide typically qualify as compact web sections and should be designed at the strength limit state using the applicable provisions of Appendix A6 instead of the more conservative provisions of Article 6.10.8. The provisions of Article A6.3.3 are also optionally applicable for the noncomposite section during construction in all regions of the beam.

For sections in regions of negative flexure in the routine multi-span continuous plate girder bridges covered by this Guide that satisfy the restrictions specified in this Article, and do not have holes in the tension flange, the optional provisions of Appendix A6 may be employed to design the section at the strength limit state; otherwise, the section is treated as a slender web section and the provisions of Article 6.10.8 must be used to design the section. Compact web sections should be designed using the provisions of Appendix A6 at the strength limit state instead of the more conservative provisions of Article 6.10.8. The provisions to use for the design of a noncompact web section in regions of negative flexure at the strength limit state are up to the judgment of the Engineer. The provisions of Article A6.3.3 may optionally be applicable for the noncomposite section during construction if the noncomposite section qualifies as a compact web or noncompact web section.

For further information on strength limit state design for flexure, consult Section 6.5.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples illustrating strength limit state design flexure checks, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS;

they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.6.3 Shear

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article simply points to the provisions of Article 6.10.9 for determining the factored shear resistance of the beam or girder at the strength limit state (see the Discussion of Article 6.10.9 in this Guide), which are applicable to the routine steel I-girder bridges covered by this Guide.

For further information on strength limit state design for shear, consult Section 6.5.7 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples illustrating strength limit state design shear checks, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.6.4 Shear Connectors

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article simply points to the provisions of Article 6.10.10.4 for determining the factored shear resistance of shear connectors for I-section flexural members at the strength limit state (see the

Discussion of Article 6.10.10.4 in this Guide), which are applicable to the routine steel I-girder bridges covered by this Guide.

For further information on strength limit state design of shear connectors, consult the Discussion of Article 6.10.10.4 in this Guide and Section 6.6.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples illustrating strength limit state design of shear connectors, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.7 Flexural Resistance—Composite Sections in Positive Flexure

6.10.7.1 Compact Sections

6.10.7.1.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article prescribes the relationship that must be satisfied at the strength limit state for compact composite I-sections in regions of positive flexure (i.e., the moment form of the one-third rule flexural resistance equation). Most composite sections in regions of positive flexure in routine steel I-girder bridges without holes in the tension flange will qualify as compact sections (see the Discussion of Article 6.10.6.2.2 in this Guide); therefore, this Article is applicable to the routine steel I-girder bridges covered by this Guide. As mentioned in the Discussion of Article 6.10.6.2.2 in this Guide, sections that qualify as compact sections may conservatively be treated as noncompact sections (see the Discussion of Article 6.10.7.2 in this Guide), if desired.

The relationship given by Eq. 6.10.7.1.1-1 includes the bottom (tension) flange lateral bending stress, f_ℓ. For the routine steel I-girder bridges covered by this Guide, the only source of flange lateral bending stress to be considered at the strength limit state is wind loading occurring under Strength load combinations that include wind load effects. f_ℓ cannot exceed $0.6F_{yf}$. Amplification of f_ℓ in the tension flange is not required. Lateral bending does not need to be considered in the top (compression) flange in regions of positive flexure because the flange is continuously supported by the concrete deck. M_u and f_ℓ are always taken as positive in sign in Eq. 6.10.7.1.1-1. However, when summing dead and live load moments to obtain the total factored major-axis moment, M_u, and total factored lateral bending stresses, f_ℓ, to apply in the equations, the signs of the individual stresses or moments must be considered. If there is no flange lateral bending considered, the f_ℓ term drops out of the equation.

For further information on strength limit state design for flexure, consult Section 6.5.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.7.1.2 Nominal Flexural Resistance

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Applicable.

Discussion:

This Article provides the equations for computing the nominal flexural resistance, M_n, of compact composite I-sections in regions of positive flexure at the strength limit state for use in checking Eq. 6.10.7.1.1-1. These equations are applicable to the multi-span continuous routine steel I-girder bridges, and partially applicable to the simple-span routine I-girder bridges, covered by this Guide for which the sections in regions of positive flexure qualify as compact (see the Discussion of Article 6.10.6.2.2 in this Guide). The nominal flexural resistance of compact sections without holes in the tension flange is permitted to exceed the moment at first yield.

There are two different equations for the nominal flexural resistance of compact composite sections in regions of positive flexure (Eq. 6.10.7.1.2-1 or Eq. 6.10.7.1.2-2), depending on the value of D_p compared with D_t. D_p is the distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment (*PNA*), and D_t is the total depth of the composite section, including the structural concrete deck. The location of the *PNA* can be determined using the provisions of Article D6.1 (see the Discussion of Article D6.1 in this Guide); for the routine steel I-girder bridges covered by this Guide, the *PNA* will most always be located either in the top flange or in the concrete deck. The haunch depth may be considered in the computation of D_t if permitted by the Owner-agency.

For simple span routine steel I-girder bridges, M_n is calculated from either Eq. 6.10.7.1.2-1 or 6.10.7.1.2-2, as applicable. The limitation given by Eq. 6.10.7.1.2-3 does not apply.

For multi-span continuous bridges, M_n computed from either Eq. 6.10.7.1.2-1 or 6.10.7.1.2-2, as applicable, cannot exceed $1.3R_hM_y$ (Eq. 6.10.7.1.2-3), where M_y is the moment at first yield determined from the provisions of Article D6.2.2 (see the Discussion of Article D6.2.2 in this Guide). For typical composite sections in positive flexure, a considerable amount of yielding and inelastic curvature is required to reach M_p. The resulting shedding of moment to adjacent interior-pier sections that do not have additional capacity to sustain these larger moments as the positive-moment section yields and loses its effective stiffness could potentially result in incremental collapse under repeated live load applications. Thus, unless the specific steps below are taken, the resistance of the section in positive flexure is conservatively limited.

The limitation given by Eq. 6.10.7.1.2-3 may be waived if the adjacent interior-pier sections satisfy the requirements given in the two bulleted items in this Article that refer to provisions given in Appendix B6 (see the Discussion of Appendix B6 in this Guide), which provide the necessary ductile moment-rotation characteristics at the interior-pier sections to sustain the larger moments. Interior-pier sections in multi-span continuous rolled beam bridges are more likely to satisfy these requirements. However, in most cases for the routine steel multi-span continuous bridges covered by this Guide, the excess flexural resistance above $1.3R_hM_y$ should not be necessary for compact composite sections in regions of positive flexure at the strength limit state, as other limit state criteria will typically control the design of the section.

For further information on strength limit state design for flexure, consult Section 6.5.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.7.2 Noncompact Sections

6.10.7.2.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article prescribes the relationships that must be satisfied at the strength limit state for the compression and tension flanges of noncompact composite I-sections in regions of positive flexure. These provisions may conditionally apply for the routine steel I-girder bridges covered by this Guide should the section not qualify as a compact section at the strength limit state, which is atypical for a routine steel I-girder bridge, or should the section qualify as a compact section and instead be conservatively designed as a noncompact section as mentioned in the Discussion of Article 6.10.6.2.2 in this Guide.

For composite I-sections in regions of positive flexure, lateral bending does not need to be considered in the top (compression) flange at the strength limit state because the flange is continuously supported by the concrete deck. However, since the bottom (tension) flange is not continuously supported, lateral bending must be considered in flexural resistance computations for the tension flange (using the stress form of the one-third rule flexural resistance equation). For the routine steel I-girder bridges covered by this Guide, the only source of flange lateral bending stress to be considered at the strength limit state is wind loading occurring under the Strength load combinations that include wind load effects. f_ℓ cannot exceed $0.6F_{yf}$. Amplification of f_ℓ in the tension flange is not required. f_{bu} and f_ℓ are always taken as positive in sign in Eq. 6.10.7.2.1-2. However, when summing dead and live load stresses to obtain the total factored major-axis stress, f_{bu}, and total factored lateral bending stresses, f_ℓ, to apply in the equations, the signs of the individual stresses must be considered. If there is no flange lateral bending considered, the f_ℓ term drops out of the equation.

This Article further limits the maximum longitudinal compressive stress in the concrete deck at the strength limit state for a noncompact I-section to $0.6f'_c$ to provide for linear behavior of the concrete, which is assumed in the calculation of the steel flange stresses. This requirement is unlikely to control for the routine steel I-girder bridges covered by this Guide.

For further information on strength limit state design for flexure, consult Section 6.5.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.7.2.2 Nominal Flexural Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article provides the equations for computing the nominal flexural resistance of the compression flange, F_{nc}, and the nominal flexural resistance of the tension flange, F_{nt}, for noncompact composite I-sections in regions of positive flexure at the strength limit state for use in checking Eqs. 6.10.7.2.1-1 and 6.10.7.2.1-2, respectively. These provisions may conditionally apply for the routine steel I-girder bridges covered by this Guide should the section not qualify as a compact section at the strength limit state, which is atypical for a routine steel I-girder bridge, or should the section qualify as a compact section and instead be conservatively designed as a noncompact section as mentioned in the Discussion of Article 6.10.6.2.2 in this Guide. The nominal flexural resistance of noncompact composite I-sections in positive flexure is limited to the moment at first yield. Therefore, the nominal flexural resistance for each flange is best expressed in terms of the flange stress for reasons discussed in the Commentary for Article 6.10.6.1 and is taken equal to the yield stress of the respective flange times the appropriate flange-strength reduction factors.

For both the compression and tension flanges, the hybrid factor, R_h (see the Discussion of Article 6.10.1.10.1 in this Guide), is applied to the flange yield stress. However, routine steel I-girder bridges as defined for the purposes of this Guide do not utilize hybrid girder designs; as a result, the hybrid factor, R_h, is to be taken equal to 1.0 and does not affect the calculation of the nominal flexural resistance.

In addition, for the compression flange, the web load-shedding factor, R_b (Article 6.10.1.10.2), is also applied to the flange yield stress. However, for the routine steel I-girder bridges covered by this Guide, R_b is to be taken equal to 1.0 and does not affect the calculation of the nominal flexural resistance because web bend buckling and subsequent load shedding are not a consideration for composite sections in regions of positive flexure in these bridges at the strength limit state (see the Discussion of Article 6.10.1.10.2 in this Guide).

For further information on strength limit state design for flexure, consult Section 6.5.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.7.3 Ductility Requirement

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The provision in this Article provides a ductility requirement that is intended to protect the concrete deck in regions of positive flexure from premature crushing. The D_p/D_t ratio is limited to 0.42 so that significant yielding of the bottom flange can be expected prior to the top of the concrete deck reaching its the crushing strain (see the Discussion of Article 6.10.7.1.2 in this Guide for the definitions of D_p and D_t). The requirement is applicable to the routine steel I-girder bridges covered by this Guide and must be checked for both compact and noncompact sections in regions of positive flexure. This requirement is unlikely to control for the routine steel I-girder bridges covered by this Guide. If it is found that this requirement is affecting the design that probably indicates a fundamental flaw in the layout of the superstructure cross-section and/or the proportioning of the girders.

6.10.8 Flexural Resistance—Composite Sections in Negative Flexure and Noncomposite Sections

6.10.8.1 General

6.10.8.1.1 Discretely Braced Flanges in Compression

Determination of applicability, *Simple Span Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

Simple Span Bridges:

This Article is not applicable to the simple span routine steel I-girder bridges covered by this Guide at the strength limit state because simple span bridges are subject to positive flexure only. The design of the discretely braced top (compression) flange of the noncomposite section during construction in these bridges is separately covered in Article 6.10.3.2.1 (see the Discussion of Article 6.10.3.2.1 in this Guide).

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

This Article provides the relationship that must be satisfied at the strength limit state for discretely braced bottom (compression) flanges of slender web sections, or compact or noncompact web sections treated as slender web sections and not designed according to the optional provisions of Appendix A6 (see the Discussion of Appendix A6 in this Guide), in regions of negative flexure in multi-span continuous steel I-girder bridges. See the Commentary for Article 6.10.6.2.3 and the Discussion of Article 6.10.6.2.3 in this Guide for further discussion on the definition and categorization of compact web, noncompact web, and slender web sections. Therefore, this Article is conditionally applicable at the strength limit state to the design of the bottom flange in regions of negative flexure in routine steel multi-span continuous I-girder bridges covered by this Guide. For composite I-sections in regions of negative flexure, lateral bending does not need to be considered in the top (tension) flange at the strength limit state because the flange is continuously supported by the concrete deck. However, since the bottom (compression) flange is discretely braced, lateral bending must be considered in flexural resistance computations for the compression flange (using the stress form of the one-third rule flexural resistance equation). The one-third rule equation when applied to compression flanges is effectively a beam-column interaction equation, expressed in terms of the flange stresses computed from elastic analysis. The terms f_{bu} and f_ℓ are analogous to the axial and bending moment terms of the beam-column interaction equation. For the routine steel I-girder bridges covered by this Guide, the only source of flange lateral bending stress to be considered at the strength limit state is wind loading occurring under the Strength load combinations that include wind load effects. Amplification of f_ℓ in the discretely braced compression flange will likely be required (see the Discussion of Article 6.10.1.6 in this Guide). f_ℓ cannot exceed $0.6F_{yf}$ after amplification. f_{bu} and f_ℓ are always taken as positive in sign in Eq. 6.10.8.1.1-1. However, when summing dead and live load stresses to obtain the total factored major-axis stress, f_{bu}, and total factored lateral bending stresses, f_ℓ, to apply in the equations, the signs of the individual stresses must be considered. If there is no flange lateral bending considered, the f_ℓ term drops out of the equation. For a discretely braced compression flange, the one-third rule equation must be checked separately for both flange local buckling and lateral-torsional buckling. When lateral-torsional buckling controls, the stress, f_{bu}, used in checking Eq. 6.10.8.1.1-1 and Eq. 6.10.3.2.1-2 is to be determined as the value of the flange compressive stress at the cross-section where $f_{bu}/R_bR_hF_{yc}$ is maximum in the unbraced length under consideration, including the end cross-sections, calculated without consideration of flange lateral bending. R_b is the *minimum* web load-shedding factor within the unbraced length under consideration, including the end cross-sections (see the Discussion of Article 6.10.1.10.2 in this Guide). R_h is the hybrid factor (see the Discussion of Article 6.10.1.10.1 in this Guide) taken equal to 1.0 since the routine steel I-girder bridges covered by this Guide do not utilize hybrid-section members. The largest value of f_ℓ within the unbraced length is also used (see the Discussion of Article 6.10.1.6 in this Guide). When flange local buckling controls, f_{bu} and f_ℓ used to check Eq. 6.10.8.1.1-1 and Eq. 6.10.3.2.1-2 may be determined as the corresponding values at the section under consideration (see the Discussion of Article 6.10.1.6 in this Guide). For the routine multi-span continuous rolled beam bridges covered by this Guide, this Article is only applicable to the discretely braced bottom flanges in regions of negative flexure at the strength limit state if the provisions of the optional Appendix A6 are not used, which is not recommended for rolled beam bridges; otherwise, this Article is not applicable.

For the routine multi-span continuous plate girder bridges covered by this Guide, this Article is applicable to the discretely braced bottom flanges in regions of negative flexure at the strength limit state if the section is a slender web section, or if the section satisfies the restrictions specified in Article 6.10.6.2.3 (see the Discussion of Article 6.10.6.2.3 in this Guide) and the provisions of the optional Appendix A6 are not used to design the section, which is not recommended for compact web sections in particular; if the provisions of Appendix A6 are used in the latter case, this Article is not applicable.

The design of the discretely braced top (compression) flange of the noncomposite section in regions of positive flexure and the discretely braced bottom (compression) flange of the noncomposite section in regions of negative flexure during construction in these bridges is covered in Article 6.10.3.2.1 (see the Discussion of Article 6.10.3.2.1 in this Guide).

For further information on strength limit state design for flexure, consult Section 6.5.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.8.1.2 Discretely Braced Flanges in Tension

Determination of applicability, *Simple Span Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

Simple Span Bridges:

This Article is not applicable to the simple span routine steel I-girder bridges covered by this Guide at the strength limit state as simple span bridges are subject to positive flexure only. The design of the discretely braced bottom (tension) flange of the noncomposite section during construction in these bridges is covered in Article 6.10.3.2.2.

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

This Article provides the relationship that must be satisfied at the strength limit state for discretely braced top (tension) flanges of slender web sections, or compact or noncompact web sections treated as slender web sections and not designed according to the optional provisions of Appendix A6 (see the Discussion of Appendix A6 in this Guide), in regions of negative flexure in multi-span continuous steel I-girder bridges. See the Commentary for Article 6.10.6.2.3 and the Discussion of Article 6.10.6.2.3 in this Guide for further discussion on the definition and categorization of compact web, noncompact web, and slender web sections. The top flange in regions of negative flexure would only be discretely braced if the shear connectors are intentionally omitted in regions of negative flexure, which is dependent on the preferences of the Owner-agency but not recommended, and if the Engineer deems that the top flange is not sufficiently encased by the concrete deck. Therefore, this Article is conditionally applicable at the strength limit state to the design of the top flange in regions of negative flexure in routine steel multi-span continuous I-girder bridges covered by this Guide.

The nominal flexural resistance, F_{nt}, in Eq. 6.10.8.1.2-1 is determined as specified in Article 6.10.8.3 (see the Discussion of Article 6.10.8.3 in this Guide) and is based on tension flange yielding. Since the top (tension) flange is discretely braced if the conditions stated above are met, lateral bending must be considered in flexural resistance computations for the tension flange (using the stress form of the one-third rule flexural resistance equation). For the routine steel I-girder bridges covered by this Guide, the only source of flange lateral bending stress to be considered at the strength limit state is wind loading occurring under the Strength load combinations that include wind load effects. f_ℓ cannot exceed $0.6F_{yf}$. Amplification of f_ℓ in the tension flange is not required. f_{bu} and f_ℓ are always taken as positive in sign in Eq. 6.10.8.1.2-1. However, when summing dead and live load stresses to obtain the total factored major-axis stress, f_{bu}, and total factored lateral bending stresses, f_ℓ, to apply in the equations, the signs of the individual stresses must be considered. If there is no flange lateral bending considered, the f_ℓ term drops out of the equation.

For the routine multi-span continuous rolled beam bridges covered by this Guide, this Article is only applicable to the discretely braced top flanges in regions of negative flexure at the strength limit state if the conditions stated above are met and the provisions of the optional Appendix A6 are not used, which is not recommended for rolled beam bridges; otherwise, this Article is not applicable.

For the routine multi-span continuous plate girder bridges covered by this Guide, this Article is applicable to the discretely braced top flanges in regions of negative flexure at the strength limit state if the conditions stated above are met and the section is a slender web section, or if the section satisfies the restrictions specified in Article 6.10.6.2.3 (see the Discussion of Article 6.10.6.2.3 in this Guide) and the provisions of the optional Appendix A6 are not used to design the section, which is not recommended for compact web sections in particular; if the provisions of Appendix A6 are used in the latter case, this Article is not applicable.

The design of the discretely braced bottom (tension) flange of the noncomposite section in regions of positive flexure and the discretely braced top (tension) flange of the noncomposite section in regions of negative flexure during construction in these bridges is covered in Article 6.10.3.2.2 (see the Discussion of Article 6.10.3.2.2 in this Guide).

For further information on strength limit state design for flexure, consult Section 6.5.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.8.1.3 Continuously Braced Flanges in Tension or Compression

Determination of applicability, *Simple Span Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Partially applicable.

Discussion:

This Article provides the relationship that must be satisfied at the strength limit state for continuously braced top (tension) flanges of slender web sections, or compact or noncompact web sections treated as slender web sections and not designed according to the optional provisions of Appendix A6 (see the Discussion of Appendix A6 in this Guide), in regions of negative flexure in multi-span continuous steel I-girder bridges. See the Commentary for Article 6.10.6.2.3 and the Discussion of Article 6.10.6.2.3 in this Guide for further discussion on the definition and categorization of compact web, noncompact web, and slender web sections. A continuously braced flange is anchored to the concrete deck by shear connectors or encased in concrete.

The provision for a continuously braced flange in compression applies only to the design of the top flange of a noncomposite section at the strength limit state in regions of positive flexure (i.e., with no shear connectors) in which the Engineer deems that the flange is sufficiently encased by the concrete deck, which is not applicable to the routine steel I-girder bridges covered by this Guide.

Simple Span Bridges:

This Article is not applicable to the routine simple span I-girder bridges covered by this Guide at the strength limit state as simple span bridges are subject to positive flexure only, and such structures are defined as having shear connectors so as to achieve a composite design in their final fully constructed condition.

The design of the continuously braced top flanges in these bridges at the strength limit state is covered in Article 6.10.7 (see the Discussion of Article 6.10.7 in this Guide).

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

For the routine multi-span continuous rolled beam bridges covered by this Guide, this Article is only applicable to a continuously braced top flange in regions of negative flexure at the strength limit state if the provisions of the optional Appendix A6 are not used, which is not recommended for rolled beam bridges; otherwise, this Article is not applicable.

For the routine multi-span continuous plate girder bridges covered by this Guide, this Article is only applicable to a continuously braced top flange in regions of negative flexure at the strength limit state if the section is a slender web section, or if the section satisfies the restrictions specified in Article 6.10.6.2.3 (see the Discussion of Article 6.10.6.2.3 in this Guide) and the provisions of the optional Appendix A6 are not used to design the section, which is not recommended for compact web sections in particular; if the provisions of Appendix A6 are used in the latter case, this Article is not applicable.

Since the flange is continuously braced, only yielding of the flange is a concern and any flange lateral bending stresses need not be considered.

For further information on strength limit state design for flexure, consult Section 6.5.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.8.2 Compression-Flange Flexural Resistance

6.10.8.2.1 General

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

This Article directs the Engineer to the Articles containing the equations necessary to compute the nominal flexural resistance, F_{nc}, of a discretely braced compression flange based on flange local buckling (see the Discussion of Article 6.10.8.2.2 in this Guide) or lateral-torsional buckling (see the Discussion of Article 6.10.8.2.3 in this Guide) for use in Eq. 6.10.8.1.1-1 at the strength limit state or in Eq. 6.10.3.2.1-2 for the noncomposite section during construction. The equations in these Articles assume the section is a slender web section whether it is or not, and the equations must be satisfied for both flange local buckling and lateral-torsional buckling. See the Commentary for Article 6.10.6.2.3 and the Discussion of Article 6.10.6.2.3 in this Guide for further discussion on the definition and categorization of compact web, noncompact web, and slender web sections.

The Commentary for this Article includes presentation of the "basic form of all I-section compression-flange flexural resistance equations." **Designers are strongly encouraged to familiarize themselves with the concepts presented in this Commentary and the associated graph in Figure C6.10.8.2.1-1. Possessing a clear understanding of these fundamental concepts is invaluable for understanding the associated provisions of the AASHTO LRFD BDS.**

Simple Span Bridges:

This Article is applicable for the simple span steel I-girder bridges covered by this Guide for determining F_{nc} for the discretely braced top (compression) flange of the noncomposite section during construction for use in Eq. 6.10.3.2.1-2; the one exception being if the section qualifies as a compact web or noncompact web section and the provisions of Article A6.3.3 are used to compute M_{nc} for lateral-torsional buckling to account for the beneficial effect of the St. Venant torsional constant, J. This Article is not applicable to these bridges at the strength limit state as simple spans are subject to positive flexure only and the top (compression) flange is continuously braced by the concrete deck.

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

For the routine multi-span continuous rolled beam bridges covered by this Guide, this Article is only applicable for determining F_{nc} for the discretely braced bottom (compression) flange in regions of negative flexure at the strength limit state for use in Eq. 6.10.8.1.1-1 if the provisions of the optional Appendix A6 (see the Discussion of Appendix A6 in this Guide) are not used, which is not recommended for rolled beam bridges; otherwise, this Article is not applicable at the strength limit state.

For the routine multi-span continuous plate girder bridges covered by this Guide, this Article is applicable for determining F_{nc} for the discretely braced bottom (compression) flange in regions of negative flexure at the strength limit state for use in Eq. 6.10.8.1.1-1 if the section is a slender web section, or if the section satisfies the restrictions specified in Article 6.10.6.2.3 (see the Discussion of Article 6.10.6.2.3 in this Guide) and the provisions of the optional Appendix A6 are not used to design the section, which is not recommended for compact web sections in particular; if the provisions of Appendix A6 are used in the latter case, this Article is not applicable at the strength limit state.

The Article is applicable for determining F_{nc} for the discretely braced top (compression) flange of the noncomposite section in regions of positive flexure and for the discretely braced bottom (compression) flange of the noncomposite section in regions of negative flexure during construction for use in Eq. 6.10.3.2.1-2; the one exception being if the section qualifies as a compact web or noncompact web section (essentially true for rolled beam sections, but not necessarily the case for plate girder sections) and the provisions of Article A6.3.3 are used to compute M_{nc} for lateral-torsional buckling to account for the beneficial effect of the St. Venant torsional constant, J.

For further information on strength limit state design for flexure, consult Section 6.5.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.8.2.2 Local Buckling Resistance

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article are used to compute the nominal flexural resistance, F_{nc}, of a discretely braced compression flange based on flange local buckling (FLB) for use in Eq. 6.10.8.1.1-2 or 6.10.3.2.1-2, as applicable.

FLB is a limit state of buckling of a compression flange within a cross-section and is a function of the compression-flange slenderness, $\lambda_f = b_{fc}/2t_{fc}$. For determining the FLB resistance (refer to Figure C6.10.8.2.1-1), λ_{pf} locates Anchor Point 1 (Eq. 6.10.8.2.2-4) that separates sections with compact flanges from sections with noncompact flanges (see Table C6.10.8.2.2-1). A member with a compression-flange slenderness at or below the compact flange limit, λ_{pf}, is able to achieve the so-called "plateau strength" or maximum potential FLB resistance (F_{max} in Figure C6.10.8.2.1-1) of $R_bR_hF_{yc}$ (Eq. 6.10.8.2.2-1), which is independent of the compression-flange slenderness. λ_{rf} locates Anchor Point 2 (Eq. 6.10.8.2.2-5) that separates sections with noncompact flanges from sections with slender flanges and is the point where the inelastic and elastic FLB resistances are the same (with the resistance at this point assumed to be R_bF_{yr}). F_{yr} for checking FLB is the compression-flange stress at the onset of nominal yielding within the cross-section, including residual stress effects, but not including compression flange lateral bending, taken as the smaller of $0.7F_{yc}$ and F_{yw}, but not less than $0.5F_{yc}$. The inelastic FLB resistance of a noncompact flange is treated as a linear function of the compression-flange slenderness (Eq. 6.10.8.2.2-2). An elastic FLB equation for slender flanges is not provided in the specifications because for most practical bridge-girder sections, including sections used in the routine I-girder bridges covered by this Guide (i.e., with $F_{yc} \leq 90$ ksi), elastic FLB will not control as λ_f is limited to a practical maximum value of 12.0 (see the Discussion of Article 6.10.2.2 in this Guide). The equations in this Article assume the section is a slender web section whether it is or not. See the Commentary for Article 6.10.6.2.3 and the Discussion of Article 6.10.6.2.3 in this Guide for further discussion on the definition and categorization of compact web, noncompact web, and slender web sections. The FLB resistance for moment gradient cases is treated the same as that for the case of uniform major-axis bending; i.e., the relatively minor influence of moment-gradient effects on the FLB resistance is neglected.

For design checks where the flexural resistance is based on FLB, f_{bu} and f_ℓ in Eqs. 6.10.8.1.1-1 and 6.10.3.2.1-2 are determined as the stress at the section under consideration (see the Discussion Article 6.10.1.6 in this Guide).

This Article is conditionally applicable for routine steel multi-span continuous I-girder bridges, and only partially applicable to routine simple span I-girder bridges covered by this Guide, as described further in the Discussion of Article 6.10.8.2.1 in this Guide (disregarding the language pointing to Article A6.3.3).

For further information on strength limit state design for flexure, consult Section 6.5.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for

Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.8.2.3 LTB Resistance

6.10.8.2.3a General

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article are used to compute the nominal flexural resistance, F_{nc}, of a slender-web section member with a discretely braced compression flange based on lateral-torsional buckling (LTB) for use in Eq. 6.10.8.1.1-2 or 6.10.3.2.1-2, as applicable (see the Discussion of Articles 6.10.8.1.1 and 6.10.3.2.1 in this Guide). The provisions address the calculation of the nominal LTB resistance for general composite or noncomposite unbraced lengths with single- or reverse curvature bending, double- or single-symmetry of the steel cross-section, and prismatic or nonprismatic geometry, including potential steps in F_{yc} along the length. The provisions are not applicable for I-section members subjected to single-curvature bending in which the flange in flexural compression is continuously braced within the entire unbraced length under consideration. For these types of unbraced lengths, the continuously braced flange is to be checked by the provisions of Article 6.10.7.1, 6.10.7.2, or 6.10.8.1.3, as applicable (see the Discussion of Articles 6.10.7.1, 6.10.7.2, and 6.10.8.1.3 in this Guide). The last paragraph of the Commentary of this Article addresses unbraced lengths in single-curvature bending where continuous bracing of a flange subjected to compression ends. For cases where a shortened unbraced length is used, points A, B, and C used in the computation of the term C_b in Article 6.10.8.2.3b (see the Discussion of Article 6.10.8.2.3b in this Guide) should be taken as the quarter points of the shortened unbraced length. Such a situation would be very uncommon in the routine steel I-girder bridges covered by this Guide. Refer to the last paragraph of this Article for special considerations for unbraced lengths subject to reverse-curvature bending.

The provisions of this Article assume slender-web behavior and are conditionally applicable for routine steel multi-span continuous I-girder bridges, and only partially applicable for routine simple span I-girder bridges covered by this Guide, as described further in the Discussion of Article 6.10.8.2.1 in this Guide. See the Commentary for Article 6.10.6.2.3 and the Discussion of Article 6.10.6.2.3 in this Guide for further discussion on the definition and categorization of compact web, noncompact web, and slender web sections.

LTB is a limit state of buckling of an unbraced length involving lateral deflection and twist and is a function of the unbraced length, L_b. For determining the LTB resistance (refer to Figure C6.10.8.2.1-1), L_p locates Anchor Point 1 that separates compact unbraced lengths from noncompact unbraced lengths (Eq. 6.10.8.2.3a-4). A member braced at or below the compact unbraced length limit is able to achieve the so-called "plateau strength" or maximum potential LTB resistance (F_{max} in Figure C6.10.8.2.1-1) of $R_bR_hF_{yc}$ under uniform vertical bending, which is independent of the unbraced length (Eq. 6.10.8.2.3a-1). Note that in many cases, it will not be economical to brace the girder at a distance equal to L_p or below to reach F_{max}, particularly under uniform bending conditions for which C_b is equal to 1.0. L_r locates Anchor Point 2 that separates sections with noncompact unbraced lengths from sections with slender unbraced lengths (Eq. 6.10.8.2.3a-5) and is the point where the inelastic and elastic LTB resistances are the same (with the resistance at this point assumed to be R_bF_{yr}). F_{yr} for checking LTB is the compression-flange stress at the onset of nominal yielding, including residual stress and geometric imperfection effects but not including compression-flange lateral bending, and is to be taken as $0.5F_{yc}$ for members with longitudinally unstiffened webs. For members with longitudinally stiffened webs, F_{yr} is to be taken as the smaller of $0.5F_{yc}$ and the web bend-buckling resistance, F_{crw} (see the Discussion of Article 6.10.1.9.1 in this Guide). The preceding values of F_{yr} provide a more uniform level of reliability consistent with the target levels in the AISC and AASHTO LRFD Specifications compared with the variable value of F_{yr}, often equal to $0.7F_{yc}$, in editions of the AASHTO LRFD BDS prior to the 10th Edition. The value of $0.7F_{yc}$ is retained for rolled-section members. The inelastic LTB resistance of a noncompact unbraced length is treated as a linear function of the unbraced length (Eq. 6.10.8.2.3a-2). Unbraced lengths greater than L_r are termed slender unbraced lengths, and their resistance is controlled by elastic LTB (Eq. 6.10.8.2.3a-3). LTB in the elastic range is of primary importance for relatively slender girders braced at longer than normal intervals, which most typically occurs during a temporary construction condition.

Article 6.10.8.2.3b applies for determining the LTB parameters C_b, F_e, and r_t to substitute in these LTB resistance equations for prismatic unbraced lengths. Article 6.10.8.2.3c applies for determining these parameters for nonprismatic unbraced lengths (see the Discussions of Articles 6.10.8.2.3b and 6.10.8.2.3c in this Guide).

In lieu of a more refined analysis, the nominal LTB resistance of a discretely braced flange subjected to flexural compression is to be calculated at the so-called "governing cross-section", which is the cross-section where $f_{bu}/R_bR_hF_{yc}$ is maximum in the unbraced length under consideration, including the end cross-sections. f_{bu} is the factored flexural compressive stress at the cross-section under consideration, calculated without considering flange lateral bending. R_b is the *minimum* web load-shedding factor within the unbraced length under consideration, including the end cross-sections (see the Discussion of Article 6.10.1.10.2 in this Guide). R_h is the hybrid factor (see the Discussion of Article 6.10.1.10.1 in this Guide) taken equal to 1.0 since the routine steel I-girder bridges covered by this Guide do not utilize hybrid-section members. The Commentary for this Article further discusses the various aspects of the determination of the governing cross-section.

For design checks where the flexural resistance is based on LTB, f_{bu} in Eqs. 6.10.8.1.1-1 and 6.10.3.2.1-2 is to be determined as the value of the flange compressive stress at the cross-section where $f_{bu}/R_bR_hF_{yc}$ is maximum in the unbraced length under consideration, including the end cross-sections, and f_ℓ is to be taken as the largest value of the stress due to lateral bending throughout the unbraced length in the flange under consideration. Combined vertical and flange lateral bending is addressed in the Specifications by effectively handling the flanges as equivalent beam-columns. The use of maximum values within the unbraced length, when the resistance is governed by member stability, i.e., LTB, is consistent with established practice in the proper application of beam-column interaction equations (see the Discussion of Article 6.10.1.6 in this Guide and the Commentary of this Article).

The effective length factor, K, for LTB is assumed equal to 1.0 in the equations of this Article. The Commentary of this Article discusses the possibility of obtaining a more refined estimate of the LTB resistance accounting for end restraint from adjacent unbraced lengths and/or connection details through the calculation of an effective unbraced length, KL_b, which may be substituted for the length, L_b, in the LTB equations of this article. References to a simple hand method for calculation of elastic LTB effective length factors are provided (see Appendix A of the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge for an example calculation using this method). Alternatively, a more refined estimate of the LTB resistance may be obtained from a direct buckling analysis (see the Commentary for Article D6.6.4 and the AASHTO Nonprismatic Girder Design Guide for guidance pertaining to this type of analysis). Typically, neither of these approaches are employed or necessary in the design of the routine steel I-girder bridges covered by this Guide. However, the application of these methods may potentially be advantageous in load rating.

The 10th Edition of the AASHTO LRFD BDS also introduced provisions and commentary related to the use of half-round I-girder bearing stiffeners at supports. The Commentary of this Article suggests that the use of half-round I-girder bearing stiffeners can provide increased torsional warping restraint, resulting in a beneficial increase in lateral-torsional buckling resistance locally in the unbraced length containing this type of stiffener, and provides simplified guidance for quantifying this beneficial effect. The use of half-round I-girder bearing stiffeners is typically reserved for cases of fairly severe skew but may be considered on some moderately skewed steel I-girders bridges when appropriate. See the Discussion of Article 6.10.11.2 and the associated sub-articles for more information on half-round I-girder bearing stiffeners.

For further information on strength limit state design for flexure, consult Section 6.5.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.8.2.3b LTB Parameters for Prismatic Unbraced Lengths

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

Article 6.10.8.2.3b provides equations for determining the LTB parameters C_b, F_e, and r_t to substitute in the LTB nominal resistance equations of Article 6.10.8.2.3a for prismatic unbraced lengths (see the Discussion of Article 6.10.8.2.3a in this Guide) of slender-web section members. A prismatic unbraced length is defined as an unbraced length between cross-frames or diaphragms in which the member cross-section and yield strength does not vary along the length.

The provisions of this Article assume slender-web behavior and are conditionally applicable for routine steel multi-span continuous I-girder bridges, and only partially applicable for routine simple span I-girder bridges covered by this Guide, as described further in the Discussion of Article 6.10.8.2.1 in this Guide. See the Commentary for Article 6.10.6.2.3 and the Discussion of Article 6.10.6.2.3 in this Guide for further discussion on the definition and categorization of compact web, noncompact web, and slender web sections.

The solid curve in Figure C6.10.8.2.1-1 is for LTB under uniform bending and is represented by the equations given in Article 6.10.8.2.3a (see the Discussion of Article 6.10.8.2.3a in this Guide). The dashed curve in Figure C6.10.8.2.1-1 shows the solid curve scaled by the moment-gradient modifier, C_b, under moment-gradient conditions, which can result in the plateau strength (F_{max}) for lateral-torsional buckling to be reached at significantly larger unbraced lengths under moment-gradient conditions when the effects of the moment gradient are included in determining the limits on the unbraced length, L_b. Refer to Article D6.4.1 for the appropriate equations to use under these conditions (i.e., the equations representing the dashed curve when C_b is calculated and is greater than 1.0), which is strongly encouraged for prismatic unbraced lengths (see the Discussion of Article D6.4.1 in this Guide).

C_b accounts for the effect of a variation in moment along the unbraced length. For prismatic unbraced lengths, C_b is to be calculated from Eq. 6.10.8.2.3b-1, which requires the input of the absolute value of the factored major-axis bending moment at one-quarter, one-half, and three-quarters of the unbraced length under consideration (i.e., points A, B, and C), respectively, calculated from the moment envelope values that produce the largest flexural compression in the flange under consideration at these points, or the smallest flexural tension in this flange if the flange is never in compression at the point. For single-curvature bending, there is no upper limit on the value of C_b computed from this equation. Since concurrent moments are normally not tracked in the analysis, it is convenient and considered acceptable to utilize the factored worst-case moments from the live load moment envelopes in conjunction with other factored moment diagrams for calculation of C_b. M_{max} in Eq. 6.10.8.2.3b-1 is the absolute value of the factored maximum major-axis moment in the unbraced length, calculated from the moment envelope value that produces the largest flexural compression in the flange under consideration. For points A, B, or C in unbraced lengths of noncomposite or composite section members where the flange under consideration is subjected to compression and is continuously braced anywhere within either quarter portion of the unbraced length adjacent to the point under consideration, the moment corresponding to that point, A, B, or C, is to be taken equal to zero in Eq. 6.10.8.2.3b-1.

The bulleted items at the beginning of the Commentary for this Article indicate the conditions for which Eq. 6.10.8.2.3b-1 is considered applicable for the computation of C_b. For prismatic noncomposite unbraced lengths of singly symmetric members subject to reverse curvature bending, Eq. 6.10.8.2.3b-1 should be multiplied by the factor, R_m, calculated from Eq. C6.10.8.2.3b-1 or C6.10.8.2.3b-2, as applicable. Note that the 1st bullet and Eq. C6.10.8.2.3b-1 only apply if there is a single point of contraflexure within the unbraced length under consideration, which is the most common situation. The resulting value of C_b from this calculation (i.e., with the R_m factor applied) is not to exceed 3.0. Alternatively, the Commentary points to the provisions of Article 6.10.8.2.3c to calculate the LTB parameters for a more refined and potentially accurate solution for prismatic noncomposite unbraced lengths of singly symmetric members subject to reverse curvature bending and for prismatic composite unbraced lengths subject to reverse curvature bending, with the elastic LTB load ratio, γ_e, calculated using Method A specified in Article D6.6.2 (see the discussion of Articles 6.10.8.2.3c and D6.6.2 in this Guide).

C_b from Eq. 6.10.8.2.3b-1 has a base value of 1.0 when the moment and the corresponding flange compressive major-axis bending stress are constant over the unbraced length. C_b may be conservatively taken equal to 1.0 for all cases, with the exception of unusual circumstances with no intermediate cross-bracing and for unbraced cantilevers with significant loading applied at the level of the top flange for which load-height effects should be considered in the calculation of C_b. The 7th and 8th paragraphs of the Commentary to this Article discuss recommendations for the handling of such cases.

Eq. 6.10.8.2.3b-2 for the elastic LTB stress, F_e, at the governing cross-section (see the discussion of Article 6.10.8.2.3a in this Guide) is a conservative approximation of the exact beam-theory solution for the elastic LTB resistance of a doubly symmetric I-section (assuming load-height effects are not considered and that the St. Venant torsional constant, J, is taken equal to zero). The radius of gyration for LTB at the governing cross-section, r_t, used in the calculation of F_e is determined from Eq. 6.10.8.2.3b-3. Alternatively, Eq. C6.10.8.2.3b-4 in the Commentary of this

Article is permitted for use for a more exact calculation of r_t for software calculations or if the Engineer requires a more precise calculation of F_e. Eq. 6.10.8.2.3b-3 is a simplification of Eq. C6.10.8.2.3b-4 obtained by taking $D = h = d$. Further details on the accuracy of these approximations in the computation of F_e are discussed in the Commentary for this Article.

As pointed out in the Commentary for this Article, the equation for F_e also neglects the restraint at the level of the tension flange provided by the lateral and torsional stiffness of the concrete deck. For I-section members with relatively large values of D/b_{fc}, D/t_w, and t_{fc}/t_w and with limited transverse web stiffening and/or web transverse stiffeners that are not attached to the top flange, the benefits of this torsional restraint on the LTB resistance tend to be small. In these cases, the influence of the torsional restraint from the deck is largely lost due to the distortional flexibility of the web. For slender-web girders, the benefits of this restraint are judged to not be worth the additional complexity associated with a general distortional buckling solution.

Simple Span Bridges:

For simple span bridges, the moment-gradient modifier, C_b, may conservatively be taken equal to 1.0 when checking LTB of the critical noncomposite unbraced length in regions of positive flexure during construction.

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

For multi-span continuous bridges, it is strongly recommended that as a minimum the moment-gradient modifier, C_b, be calculated when checking LTB of the first unbraced length on either side of the interior piers, specifically for prismatic unbraced lengths or for nonprismatic unbraced lengths satisfying the 20 percent rule described below in the discussion of Article 6.10.8.2.3c in this Guide. The unbraced lengths on either side of the pier should be checked to determine which side will yield the lower value of C_b..

For multi-span continuous rolled beam bridges, if the provisions of the optional Appendix A6 are not used, which is not recommended for rolled beam bridges, C_b should be calculated for the first unbraced length on either side of the interior piers using Eq. 6.10.8.2.3b-1.The provisions of Article D6.4.1 should then be employed to determine the shift in the anchor point, L_p, and the corresponding nominal LTB resistance for these unbraced lengths (see the Discussion of Article D6.4.1 in this Guide).

For multi-span continuous steel plate girder bridges, if the section is a slender web section or if the section satisfies the restrictions specified in Article 6.10.6.2.3 (see the Discussion of Article 6.10.6.2.3 in this Guide) and the provisions of the optional Appendix A6 are not used to design the section, which is not recommended for compact web sections in particular, C_b should be calculated for the first unbraced length on either side of the interior piers using Eq. 6.10.8.2.3b-1.The provisions of Article D6.4.1 should then be employed to determine the shift in the anchor point, L_p, and the corresponding nominal LTB resistance for these unbraced lengths (see the Discussion of Article D6.4.1 in this Guide).

C_b may conservatively be taken equal to 1.0 when checking LTB of the critical noncomposite unbraced length in regions of positive flexure in multi-span continuous bridges during construction.

For further information on strength limit state design for flexure, consult Section 6.5.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.8.2.3c LTB Parameters for Nonprismatic Unbraced Lengths

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

Article 6.10.8.2.3c provides equations for determining the LTB parameters C_b, F_e, and r_t to substitute in the LTB nominal resistance equations of Article 6.10.8.2.3a for nonprismatic unbraced lengths of slender-web section members (see the Discussion of Article 6.10.8.2.3a in this Guide). A nonprismatic unbraced length is defined as an unbraced length between cross-frames or diaphragms in which the member cross-section and/or yield strength varies along the length. The provisions of Article D6.4.1 (see the Discussion of Articles 6.10.8.2.3b and D6.4.1 in this Guide) are not to be employed for nonprismatic unbraced lengths.

The provisions of this Article assume slender-web behavior and are conditionally applicable for routine steel multi-span continuous I-girder bridges, and only partially applicable for routine simple span I-girder bridges covered by this Guide, as described further in the Discussion of Article 6.10.8.2.1 in this Guide. See the Commentary for Article 6.10.6.2.3 and the Discussion of Article 6.10.6.2.3 in this Guide for further discussion on the definition and categorization of compact web, noncompact web, and slender web sections.

The Article presents provisions to directly calculate the LTB resistance of nonprismatic unbraced lengths. However, the application of those provisions is fairly involved and complicated, so the Article also allows for treatment of a nonprismatic unbraced length as prismatic if certain geometric conditions and proportional limits are met. For new design of the routine steel I-girder bridges covered by this Guide, it is appropriate, reasonable, and recommended to configure the design of the girders to meet the requirements that allow for simplified treatment of a nonprismatic unbraced length as prismatic. The more complicated and involved procedure for directly calculating the LTB resistance of nonprismatic unbraced lengths should be reserved for load rating of existing structures or the design of non-routine bridges.

In this Discussion, the provisions that allow for treatment of a nonprismatic unbraced length as prismatic are presented first. For unbraced lengths of constant web depth members containing a single cross-section transition to a smaller section at a distance less than or equal to 20 percent of the unbraced length from the brace point with the smaller magnitude of the moment, and where the larger magnitude of the moment occurs within the section with the largest resistance, and where the ratios of the lateral moments of inertia, I_{yt1}/I_{yt2} and I_{yb1}/I_{yb2}, of the adjacent top and bottom flanges, respectively, of each section at the transition are greater than or equal to 0.5 (where Flange 1 is the flange located closer to the brace point with the smaller magnitude of moment and Flange 2 is the flange located closer to the larger magnitude of moment), the LTB resistance of the compression flange, F_{nc}, may be determined assuming the unbraced length is prismatic using the parameters calculated as specified in Article 6.10.8.2.3b (see the Discussion of Article 6.10.8.2.3b in this Guide); that is, assuming that the transition does not exist and that the flanges of the section closer to the brace point with the larger magnitude of moment are extended to the brace point with the smaller magnitude of moment.

For the case of uniform bending, the reduction in the elastic LTB resistance due to a cross-section transition located within the unbraced length of a constant web depth member is approximately five percent when the transition is placed with 20 percent of the unbraced length from the brace point with the smaller magnitude of moment and the preceding flange moment of inertia requirements are satisfied. The moment gradient modifier, C_b, from Article 6.10.8.2.3b should be calculated and applied in this case (see the Discussion of Article 6.10.8.2.3b in this Guide) and L_b may also be modified by an effective length factor, if desired (see the Discussion of Article 6.10.8.2.3a in this Guide). Otherwise, the LTB resistance of the unbraced length is to be determined as specified in this Article.

For a constant web depth member with more than one transition, all transitions located at or closer than 20 percent of the unbraced length from the brace point with the smaller magnitude of moment, and with the ratio of the lateral moments of inertia, I_{yt1}/I_{yt2} and I_{yb1}/I_{yb2}, of the adjacent flanges of each section (i.e., $t_f b_f^3/12$) equal to or larger than 0.5, may be ignored. In such cases, the LTB resistance of the remaining prismatic or nonprismatic unbraced length may then be computed as specified in Article 6.10.8.2.3b or Article 6.10.8.2.3c, as applicable, based on the remaining sections. In addition, any adjacent section transitions, involving stepping the thickness of the web or the area of the flanges, closer than 25 percent of the unbraced length from each other should be considered to all be a part of the same section transition. The “equivalent” single section transition should be taken as located at the flange transition furthest from the closest brace point. Where a cross-section transition within the unbraced length occurs at a bolted field splice, refer to this Article regarding the calculation of I_{yt1} and I_{yb1}, the location of the transition, and the minimum length and contribution of the flange splice plates.

For nonprismatic unbraced lengths with transitions located further than 20 percent of the unbraced length from the brace point with the smaller magnitude of moment, or not satisfying the previously mentioned moment of inertia proportioning requirements, this Article includes provisions for the direct calculation of the LTB resistance of the nonprismatic unbraced length. In this Article, an equivalent r_t for LTB at the governing cross-section (see the discussion of Article 6.10.8.2.3a in this Guide) is computed from Eq. 6.10.8.2.3c-3, which allows for the direct use of Eqs. 6.10.8.2.3a-1 through 6.10.8.2.3a-3 to compute the nominal LTB resistance of the nonprismatic unbraced length. The equivalent r_t is computed from the elastic LTB stress, F_e, determined from Eq. 6.10.8.2.3c-2. F_e is determined as the product of f_{bu}, which is to be taken as the value of the flange compressive stress at the cross-section where $f_{bu}/R_bR_hF_{yc}$ is maximum in the unbraced length under consideration, including the end cross-sections (i.e., the governing cross-section), and an elastic LTB load ratio, γ_e, which is a constant by which the calculated design moments and stresses at the governing cross-section would need to be scaled to reach the theoretical elastic LTB load level (refer to Figure C6.10.8.2.3c-1 in the Commentary of this Article). R_b is the *minimum* web load-shedding factor within the unbraced length under consideration, including the end cross-sections (see the Discussion of Article 6.10.1.10.2 in this Guide). R_h is the hybrid factor (see the Discussion of Article 6.10.1.10.1 in this Guide) taken equal to 1.0 since the routine steel I-girder bridges covered by this Guide do not utilize hybrid-section members.

The elastic LTB load ratio, γ_e, may be calculated using one of the three alternative methods specified in Article D6.6 (i.e., Method A, Method B, or Method C - see the discussion of Article D6.6 in this Guide). There is no particular favor given to any of the alternative methods to calculate γ_e. The designer is free to evaluate each method and choose which one is easier to use, better suited to the situation at hand, etc. The methods should give reasonably comparable results in most cases. The more approximate Methods A and B were determined to be viable and are just different approaches to investigate a very complex problem in a reasonable fashion. The methods do not supersede each other. For cases where the elastic LTB resistance is calculated directly from an elastic eigenvalue buckling analysis (i.e., Method C), γ_e is to be taken as the smallest, or controlling, eigenvalue obtained from the buckling solution. Since moment-gradient effects are directly considered in the computation of γ_e by Methods A, B, or C, the moment-gradient modifier, C_b, is to be taken equal to 1.0 (Eq. 6.10.8.2.3c-1) whenever the provisions of Article 6.10.8.2.3c are employed to compute the nominal LTB resistance.

For further information and design examples illustrating the application of the LTB provisions for nonprismatic unbraced lengths using Methods A, B, and C, consult the AASHTO Nonprismatic Girder Design Guide. For further information on strength limit state design for flexure, consult Section 6.5.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.8.3 Flexural Resistance Based on Tension Flange Yielding

Determination of applicability, *Simple Span Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

Simple Span Bridges:

This Article is not applicable to the routine simple span I-girder bridges covered by this Guide at the strength limit state because simple span bridges are subject to positive flexure only.

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

The provisions of this Article are used to compute the nominal flexural resistance, F_{nt}, of a discretely braced top (tension) flange at the strength limit state in regions of negative flexure in multi-span continuous steel I-girder bridges for use in Eq. 6.10.8.1.2-1 (see the Discussion of Article 6.10.8.1.2 in this Guide). The nominal flexural resistance is based only on nominal yielding because flange local buckling and lateral-torsional buckling are not a consideration for flanges in tension. The equation in this Article assumes the section is a slender web section whether it is or not. See the Commentary for Article 6.10.6.2.3 and the Discussion of Article 6.10.6.2.3 in this Guide for further discussion on the definition and categorization of compact web, noncompact web, and slender web sections.

The top flange in regions of negative flexure would only be discretely braced if the shear connectors are intentionally omitted in regions of negative flexure, which is dependent on the preferences of the Owner-agency but not recommended, and if the Engineer deems that the top flange is not sufficiently encased by the concrete deck. Therefore, this Article is conditionally applicable at the strength limit state to the design of the top flange in regions of negative flexure in routine steel multi-span continuous I-girder bridges, as described further in the Discussion of Article 6.10.8.1.2 in this Guide.

For sections in which $M_{yt} > M_{yc}$, Eq. 6.10.8.3-1 does not control and tension flange yielding need not be checked, where M_{yc} and M_{yt} are the yield moments with respect to the compression and tension flange, respectively, determined as specified in Article D6.2 (see the Discussion of Article D6.2 in this Guide).

For further information on strength limit state design for flexure, consult Section 6.5.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.9 Shear Resistance

6.10.9.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article provides general provisions related to the determination of the factored shear resistance of a beam or girder, which are partially applicable to the routine steel I-girder bridges covered by this Guide.

The provisions in this Article dealing with hybrid I-shaped members and web longitudinal stiffeners are not applicable to the routine steel I-girder bridges covered by this Guide.

The general design equation for shear is given by Eq. 6.10.9.1-1. A helpful flowchart to guide designers through the design provisions for the shear design of I-sections is given in Figure C6.10.9.1-1.

This Article also provides the maximum stiffener spacing requirements for interior and end web panels of beams or girders containing web transverse stiffeners. Interior web panels with transverse stiffener spacings exceeding 3D, where D is the web depth, are to be considered unstiffened. End web panels with transverse stiffener spacings exceed 1.5D are to be considered unstiffened. An end panel is defined as a web panel adjacent to a discontinuous end of a girder. Web panels over the interior piers of a continuous span are classified as interior web panels and should not be classified as end panels. For the routine steel rolled-beam bridges covered by this Guide, web transverse stiffeners are typically not needed, except for use as cross-frame or diaphragm connection plates. When web transverse stiffeners are required, they are to be designed according to the provisions of Article 6.10.11.1.

To find a suitable stiffener spacing, the designer should select a stiffener spacing that is less than the specified maximum spacing described above and that satisfies the general shear equation (Eq. 6.10.9.1-1) using the maximum factored shear, V_u, in the web panel under consideration at the strength limit state and the applicable nominal shear resistance, V_n, (Article 6.10.9.2 or 6.10.9.3). This can sometimes involve a trial-and-error process. It is often helpful to use constant stiffener spacings over specified ranges. Since the design shear varies over the length of the girder, this procedure may have to be repeated for several ranges along the length of the girder. Cross-frame or diaphragm connection plates can be considered to act as transverse stiffeners. Therefore, ranges between connection plates are typically investigated in laying out the stiffeners.

The AASHTO-NSBA Steel Bridge Collaboration Guideline G12.1-2020 Guidelines to Design for Constructability and Fabrication provides practical guidance for economical proportioning of girder webs, such as the general preference of fabricators for a minimum web thickness of ½" to reduce the deformation of the web and the potential for weld defects during fabrication.

Changes in the web thickness along the girder in plate-girder bridges preferably should be made at field splices. In field sections over interior piers in continuous spans, the web thickness may have to be increased (typically in 1/16-inch increments) over the thickness provided in adjacent regions in positive flexure in some instances; e.g., if the web bend-buckling resistance is exceeded in regions of negative flexure at the service limit state (see the Discussion of Articles 6.10.1.9.1 and 6.10.4.2.2 in this Guide).

A useful guideline for determining the trade-off between adding more transverse stiffeners versus increasing the thickness of the web material in routine plate-girder bridges is that approximately 4 to 5 pounds of web material should be saved for every 1 pound of stiffener material added. This higher unit cost reflects that additional fabrication effort is required per pound of stiffener still than per pound of girder web steel. Generally, an unstiffened web is not the most economical alternative for a plate-girder bridge. The best solution usually includes some transverse stiffeners over the piers and near the abutments; a so-called partially stiffened web. However, it should be pointed out that a partially stiffened web can, and probably will, include a number of unstiffened panels. This is a natural outcome of an economical web design.

For further information on strength limit state design for shear, consult Section 6.5.7 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.9.2 Nominal Resistance of Unstiffened Webs

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The provisions of this Article are used to calculate the nominal shear resistance, V_n, of unstiffened web panels (i.e., interior web panels with transverse stiffener spacings exceeding 3D, or end web panels with transverse stiffener spacings exceeding 1.5D), at the strength limit state. The provisions are applicable to the routine steel I-girder bridges covered by this Guide; in particular, they are typically used to determine the nominal shear resistance for a rolled-beam web, and to determine the regions where transverse stiffeners are required in a stiffened plate-girder web.

For an unstiffened web panel, the nominal shear resistance is based on either the shear yield or shear buckling resistance (Eq. 6.10.9.2-1). C in Eq. 6.10.9.2-1 is the ratio of the shear-buckling resistance to the shear yield strength. Its value depends on the web slenderness, and the equations used to calculate C are given in Article 6.10.9.3.2. The plastic shear force, V_p (Eq. 6.10.9.2-2), is equal to the web area times the assumed shear yield strength. The assumed shear yield strength is the web yield strength divided by the square root of 3, or 0.58 (based on the von Mises yield criterion). When C equals 1.0, the nominal shear resistance is based on shear yielding. Otherwise, the nominal shear resistance is based on shear buckling. For unstiffened web panels, the shear-buckling coefficient, k, is always taken equal to 5.0 in the computation of C.

A helpful flowchart to guide designers through the design provisions for the shear design of I-sections is given in Figure C6.10.9.1-1.

The AASHTO-NSBA Steel Bridge Collaboration Guideline G12.1-2020 Guidelines to Design for Constructability and Fabrication provides practical guidance for economical proportioning of girder webs, such as the general preference of fabricators for a minimum web thickness of ½" to reduce the deformation of the web and the potential for weld defects during fabrication.

Changes in the web thickness along the girder in plate-girder bridges preferably should be made at field splices. In field sections over interior piers in continuous spans, the web thickness may have to be increased (typically in 1/16-inch increments) over the thickness provided in adjacent regions in positive flexure in some instances; e.g., if the web bend-buckling resistance is exceeded in regions of negative flexure at the service limit state (see the Discussion of Articles 6.10.1.9.1 and 6.10.4.2.2 in this Guide).

A useful guideline for determining the trade-off between adding more transverse stiffeners versus increasing the thickness of the web material in routine plate-girder bridges is that approximately 4 to 5 pounds of web material should be saved for every 1 pound of stiffener material added. This higher unit cost reflects that additional fabrication effort is required per pound of stiffener still than per pound of girder web steel. Generally, an unstiffened web is not the most economical alternative for a plate-girder bridge. The best solution usually includes some transverse stiffeners over the piers and near the abutments; a so-called partially stiffened web. Transverse stiffeners (other than connection plates for cross-frames or diaphragms) are typically not required in routine rolled-beam bridges. However, it should be pointed out that a partially stiffened web can, and probably will, include a number of unstiffened panels. This is a natural outcome of an economical web design.

For further information on strength limit state design for shear, consult Section 6.5.7 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.9.3 Nominal Resistance of Stiffened Webs

6.10.9.3.1 General

Determination of applicability, *Simple Span Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Applicable.

Discussion:

This Article points to the Articles containing the equations used to calculate the nominal shear resistance, V_n, of stiffened web panels (i.e., interior web panels with transverse stiffener spacings not exceeding $3D$, or end web panels with transverse stiffener spacings not exceeding $1.5D$), at the strength limit state. The provisions are applicable to the routine steel plate girder bridges covered by this Guide (simple span and multi-span continuous) and are not applicable to the routine steel rolled beam bridges covered by this Guide (simple span or multi-span continuous), as web transverse stiffeners are typically not needed on rolled beam bridges except for use as cross-frame or diaphragm connection plates.

The nominal shear resistance of transversely stiffened interior web panels is taken as the shear-yielding resistance or the sum of the shear-buckling resistance and the post-buckling shear resistance due to tension-field action (Article 6.10.9.3.2). The nominal shear resistance of transversely stiffened end web panels is taken as the shear-yielding or shear-buckling resistance (Article 6.10.9.3.3). Web longitudinal stiffeners are not applicable to the routine steel I-girder bridges covered by this Guide.

A helpful flowchart to guide designers through the design provisions for the shear design of I-sections is given in Figure C6.10.9.1-1.

The AASHTO-NSBA Steel Bridge Collaboration Guideline G12.1-2020 Guidelines to Design for Constructability and Fabrication provides practical guidance for economical proportioning of girder webs, such as the general preference of fabricators for a minimum web thickness of ½" to reduce the deformation of the web and the potential for weld defects during fabrication.

Changes in the web thickness along the girder in plate-girder bridges preferably should be made at field splices. In field sections over interior piers in continuous spans, the web thickness may have to be increased (typically in 1/16-inch increments) over the thickness provided in adjacent regions in positive flexure in some instances; e.g., if the web bend-buckling resistance is exceeded in regions of negative flexure at the service limit state (see the Discussion of Articles 6.10.1.9.1 and 6.10.4.2.2 in this Guide).

A useful guideline for determining the trade-off between adding more transverse stiffeners versus increasing the thickness of the web material in routine plate-girder bridges is that approximately 4 to 5 pounds of web material should be saved for every 1 pound of stiffener material added. This higher unit cost reflects that additional fabrication effort is required per pound of stiffener still than per pound of girder web steel. Generally, an unstiffened web is not the most economical alternative for a plate-girder bridge. The best solution usually includes some transverse stiffeners over the piers and near the abutments; a so-called partially stiffened web. However, it should be pointed out that a partially stiffened web can, and probably will, include a number of unstiffened panels. This is a natural outcome of an economical web design.

It should be noted that cross-frame and diaphragm connection plates also function as transverse stiffeners. Strategic determination of the cross-frame spacing can sometimes make a significant difference in the web design of a steel plate girder, particularly in longer spans where the addition of one more line of cross-frames or diaphragms may result in an effective stiffener spacing less than 3*D* without having to provide additional transverse stiffeners. Other strategies may include using a tighter cross-frame or diaphragm spacing nearer to the interior supports, and wider spacing near mid-span, of multi-span continuous steel plate girder bridges, again with the goal of providing an effective stiffener spacing less than or equal to 3*D* without having to provide additional transverse stiffeners in higher shear regions. These strategies should be used judiciously, recognizing the impact of such changes on the flexural resistance of the girder and on the overall cost of the steel superstructure.

For further information on strength limit state design for shear, consult Section 6.5.7 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.9.3.2 Interior Panels

Determination of applicability, *Simple Span Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Applicable.

Discussion:

The provisions of this Article are used to calculate the nominal shear resistance, V_n, of a stiffened interior web panel, or a web panel not adjacent to a discontinuous end of a girder with a transverse stiffener spacing not exceeding $3D$. Web panels over the interior piers of a continuous span are classified as interior web panels and should not be classified as end panels. The provisions are applicable to the routine steel plate girder bridges covered by this Guide (simple span and multi-span continuous). The provisions are partially applicable to the routine steel rolled beam bridges covered by this Guide (simple span or multi-span continuous), because in those cases the provisions in this Article are only used for these bridges to calculate the constant C to be used in determining V_n of the unstiffened web (Article 6.10.9.2) using a shear-buckling coefficient, k, equal to 5.0.

For an interior panel of a stiffened plate-girder web, the equation to use to calculate V_n depends on the ratio of the average area of the flanges within the panel relative to the area of the web (Eq. 6.10.9.3.2-1). If the inequality in this equation is satisfied, the panel can develop the full post-buckling shear resistance due to tension-field action (i.e., Eq. 6.10.9.3.2-2 is used).

Eq. 6.10.9.3.2-2 includes two components; the shear yield or shear buckling resistance, and the post-buckling tension-field resistance. The shear yield or shear buckling resistance component is identical to the equation used previously for unstiffened webs (Eq. 6.10.9.2-1); shear in this case is assumed to be carried by “beam action”. The second component is the post-buckling tension-field resistance. This component is analogous to the tension diagonals of a Pratt truss; that is, after buckling the tension forces are resisted by membrane action of the web while the compression forces are resisted by the transverse stiffeners in combination with the adjacent portions of the web. The total shear resistance is either the shear yield resistance (i.e. the second term in the equation goes to zero when C = 1.0), or the sum of the shear buckling resistance and the post-buckling resistance due to tension-field action. The constant C for use in Eq. 6.10.9.3.2-2 is calculated from Eq. 6.10.9.3.2-4, 6.10.9.3.2-5, or 6.10.9.3.2-6, as applicable, using the shear buckling coefficient, k, given by Eq. 6.10.9.3.2-7 which is dependent on the stiffener spacing, d_o.

When Eq. 6.10.9.3.2-1 is not satisfied, the average area of the flanges within the panel is small relative to the area of the web and the full post-buckling resistance generally cannot be developed. In this case, V_n is calculated from Eq. 6.10.9.3.2-8. This shear resistance equation is based on a lesser level of the post-buckling resistance, which neglects the increase in stress within the wedges of the web panel outside of the tension band implicitly included in the tension-field model. The equation is similar in form to Eq. 6.10.9.3.2-2, except that this equation has an extra d_o/D term in the denominator.

See also the Discussion of Article 6.10.9.3.1 in this Guide for further explanation of the basic concepts behind the shear design provisions, helpful design tips, handy references to other guideline documents and design examples, and comments on design software available to help automate the shear design checks.

6.10.9.3.3 End Panels

Determination of applicability, *Simple Span Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Applicable.

Discussion:

The provisions of this Article are used to calculate the nominal shear resistance, V_n, of a stiffened end web panel, or a web panel adjacent to a discontinuous end of a girder with a transverse stiffener spacing not exceeding 1.5D. Web panels over the interior piers of a continuous span are classified as interior web panels and should not be classified as end panels. The provisions are applicable to the routine steel plate girder bridges covered by this Guide (simple span and multi-span continuous) and are partially applicable to the routine steel rolled beam bridges covered by this Guide (simple span or multi-span continuous), as the provisions in this Article are only used for these bridges to calculate the constant C to be used in determining V_n of the unstiffened web end panel (Article 6.10.9.2) using a shear-buckling coefficient, k, equal to 5.0.

For the end panels of a stiffened web, V_n is based on either shear yield or shear buckling. The same equation is used to calculate V_n that is used for an unstiffened web panel (Eq. 6.10.9.2-1). However, in this case, the shear-buckling coefficient, k (Eq. 6.10.9.3.2-7), used in determining C is based on the spacing from the support to the first stiffener adjacent to the support, which cannot exceed

1.5D. The shear in stiffened end panels is limited to either the shear-yield or shear-buckling resistance to provide an anchor for the tension field in adjacent stiffened interior panels. In other words, it absorbs any imbalance of the computed horizontal component of the diagonal tensile stress in the adjacent panels.

It may initially seem counterintuitive to recognize tension field action in the calculation of the shear resistance of interior panels but not in end panels, as if the full resistance of the web is being discounted at the point where the applied shear is greatest. However, keep in mind that by limiting the transverse stiffener spacing to 1.5D in end panels, those panels can develop a higher initial shear buckling resistance, comparable to or greater than the sum of the shear buckling and post-buckling resistance of the interior panels.

See also the Discussion of Article 6.10.9.3.1 in this Guide for further explanation of the basic concepts behind the shear design provisions, helpful design tips, handy references to other guideline documents and design examples, and comments on design software available to help automate the shear design checks.

6.10.10 Shear Connectors

6.10.10.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article provides general discussion on the purpose of shear connectors and the provision of shear connectors throughout different regions of the span length. Only the provisions dealing with straight continuous composite bridges are applicable for the routine steel I-girder bridges covered by this Guide.

In general, in the absence of contradicting Owner-agency policy, it is recommended for the routine steel I-girder bridges covered by this Guide that shear connectors be provided throughout the length of the bridge, including in regions of negative flexure in multi-span continuous bridges, because doing so helps to better control cracking of the deck in regions of negative flexure. Shear connectors must be provided in these regions where the longitudinal deck reinforcement is considered in the computation of the composite section properties, which is recommended to allow for the use of a slightly smaller top flange than bottom flange in these regions. The provision of shear connectors in these regions also allows the concrete deck to be considered effective in tension at the fatigue and service limit states if other requirements are satisfied (see the Discussion of Articles 6.6.1.2.1 and 6.10.4.2.1 in this Guide). If shear connectors are omitted in these regions, which depends on the preferences of the Owner-agency but is not recommended, other provisions related to the shear connectors (see the Discussion of Article 6.10.10.3 in this Guide) and longitudinal reinforcing in the deck (see the Discussion of Article 6.10.1.7 in this Guide) apply.

6.10.10.1.1 Types

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Articles deals with the types of shear connectors permitted and the permissible proportions of a stud-type shear connector. Only stud-type shear connectors are covered in the AASHTO LRFD BDS The ratio of the total height to the shank diameter of a stud shear connector is not to be less than 5.0 when embedded in normal-weight concrete and 7.0 when embedded in lightweight concrete. Analysis of shear connector strength tests found that once the total height-to-shank diameter ratio of a stud is 5.0 or greater for normal-weight concrete or 7.0 or greater for lightweight concrete, the shear resistance of a stud shear connector is controlled solely by a proportion of the connector tensile strength (see the Discussion of Article 6.10.10.4.3 in this Guide).

6.10.10.1.2 Pitch

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

The provisions of this Article are used to determine the pitch, *p*, of stud shear connectors to satisfy the fatigue limit state and are partially applicable to the routine steel I-girder bridges covered by this Guide as described further below. The resulting number of shear connectors is then checked against the number required to satisfy the strength limit state (see the Discussion of Article 6.10.10.4 in this Guide). The routine steel I-girder bridges covered in this Guide utilize regularly spaced shear connectors that are placed in approximately equidistant intervals within one or more regions along the longitudinal axis of the beam or girder. A different pitch requirement applies to shear connector clusters, which are repeated groupings of three or more equally spaced rows of shear stud connectors spaced at intervals along the longitudinal axis of the beam or girder. Shear connector clusters are typically used when full-depth precast deck panels are utilized in accelerated bridge construction applications with the clustered shear connectors placed in grouted pockets within the panels. Research indicated an increased shear demand in the outer rows of clustered configurations that required an adjustment to the pitch requirement for regularly spaced connectors given by Eq. 6.10.10.1.2-2. Full-depth precast deck panels are not utilized in the routine steel I-girder bridges covered by this Guide.

Only the longitudinal fatigue shear range, V_{fat}, needs to be considered in determining the horizontal fatigue shear range, V_{sr}, when designing shear connectors for the routine steel I-girder bridges covered in this Guide. The radial fatigue shear range, F_{fat}, is intended to reflect the effects of torsion in the girders due to curvature, significant skew, or discontinuous cross-frame or diaphragm lines; for the purposes of this Guide, routine steel I-girder bridges are defined as not having any of these characteristics, and so it is not necessary to consider the radial fatigue shear range, F_{fat}.

The vertical shear range, V_f, used to calculate V_{fat} is determined using the fatigue live load (see the Discussion of Article 3.6.1.4 in this Guide) shears. The fatigue live load is placed in a single lane with a dynamic load allowance of 15 percent applied (Table 3.6.2.1-1). The shear range is the algebraic difference of the maximum and minimum live load plus impact shears; for a simple span, the minimum live load plus impact shear is zero.

The shears are factored for the Fatigue I load combination (Table 3.4.1-1) when the 75-year single lane Average Daily Truck Traffic $(ADTT)_{SL}$ (see the Discussion of Article 3.6.1.4.2 in this Guide) is greater than or equal to the 75-year $(ADTT)_{SL}$ Equivalent to Infinite life of 11,320 trucks per day (specified for Condition 9.2 in Table 6.6.1.2.3-1). Otherwise, the shears are factored for the Fatigue II load combination. For a fatigue design life other than 75 years and/or a number of stress cycles per truck passage (n from Table 6.6.1.2.5-1) other than 1.0, see the Commentary for Article 6.6.1.2.3. It is recommended that the moment of inertia, I, and first moment of the deck area, Q, used to calculate V_{fat} be computed using the short-term composite section (see the Discussion of Article 6.10.1.1.1b in this Guide). See the Discussion of Article 6.10.10.2 in this Guide for the calculation of the shear fatigue resistance, Z_r, of an individual stud shear connector.

The pitch can, and should, vary along the length of the girder. Typically, the calculation of the required pitch is performed at uniformly spaced points along the length of the girder (e.g., at 1/10th points or at 1/20th points, etc.), and then the specified pitch is determined over various regions of the girder such that the specified pitch within the region is less than or equal to the required pitch at the point (or points) under consideration within that region. The lengths of each region do not need to be equal; in general, smaller regions are used where the required pitch is tighter and changing at an increased rate along the length of the girder, typically in areas near supports where the shear is largest. The specified pitch (in even-inch increments) within each region is generally conservatively adjusted to produce a number of spaces that sums to the length of the region. The number of regions can be determined at the discretion of the designer. For regularly spaced shear connectors, the pitch in each region must not be less than four stud diameters for all web depths and must not exceed 48.0 inches (or 24.0 inches if the web depth is less than 24.0 inches). In general, the use of an excessive number of regions is discouraged as it rarely saves a significant number of shear connectors. Likewise, using only one or a very few regions is also discouraged as it would result in providing an excessive number of shear connectors.

For more explanation and examples of the determination of the design of shear connectors at the fatigue and strength limit states, see Section 6.3.6.3 of Reference Manual for NHI Course 130122, Design and Evaluation of Steel Bridges for Fatigue and Fracture, Section 6.6.2 of Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures as well as NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge.

The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It performs design calculations addressing the demand on, and resistance of, shear connectors at the fatigue and strength limit states in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.10.1.3 Transverse Spacing

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article provides the transverse spacing requirements (i.e., across the width of the top flange) and the clear edge distance requirements for shear connectors. The provisions of this Article are applicable to the routine steel I-girder bridges covered by this Guide. Typically, the transverse spacing of shear connectors is held constant along the entire length of a girder for simplicity of detailing and fabrication.

See also the Discussion of Articles 6.10.10.1.2 and 6.10.10.4.2 in this Guide for related explanations of shear connector design.

6.10.10.1.4 Cover and Penetration

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article provides the cover and penetration requirements for shear connectors and is applicable to the routine steel I-girder bridges covered by this Guide. It is recommended that the specified cover and penetration minimums be shown in the contract documents, with the stud height to be detailed based on field measurements of the actual beam or girder elevations.

These provisions only require that shear connectors be detailed to penetrate 2 in. into the concrete deck. In most cases, the height of the shear connectors is detailed so that they penetrate above the bottom mat of deck reinforcing. Shear connectors on members in structures that are not load-path redundant must be detailed to penetrate at least above the bottom mat of deck reinforcement to provide additional ductility. The routine steel-I-girder bridges covered by this Guide are all load-path redundant. The girders in the routine steel I-girder bridges covered by this Guide are typically plumb after deck placement, but the deck will likely have a cross-slope. This should be accounted for when detailing the height of the shear connectors; shear connectors closer to one edge of the top flange will likely have a different penetration than those closer to the other edge. In addition, for plate-girder bridges, the haunch dimension is typically measured from the top of the web (or bottom of the top flange) and the top-flange thickness may vary along the length of the girder. This should also be accounted for when detailing the height of the shear connectors.

Regardless of these variations in penetration due to deck cross slope or top-flange thickness changes, the use of different shear connector heights along the length of a girder or across the width of the top flange is rarely necessary and is discouraged because it adds unnecessary cost and complexity to the fabrication of the girders. Instead, a shear connector height should be detailed that provides at least the minimum 2 in. penetration into the deck and provides at least the minimum required cover to the top of the deck and allow the penetration into the deck to vary as needed within the range defined by those two criteria.

6.10.10.2 Fatigue Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The provisions of this Article are used to determine the fatigue resistance of an individual stud shear connector, Z_r. Z_r is used in the calculation of the required pitch, *p*, at the fatigue limit state (see the Discussion of Article 6.10.10.1.2 in this Guide). Z_r is equal to the nominal fatigue resistance, $(\Delta F)_n$, determined as specified in Article 6.6.1.2.5 (see the Discussion of Article 6.6.1.2.5 in this Guide) using the applicable values specified for Condition 9.2 in Table 6.6.1.2.3-1, times the cross-sectional area of the stud shear connector.

When the 75-year single lane Average Daily Truck Traffic $(ADTT)_{SL}$ (see the Discussion of Article 3.6.1.4.2 in this Guide) is greater than or equal to the 75-year $(ADTT)_{SL}$ Equivalent to Infinite life of 11,320 trucks per day (specified for Condition 9.2 in Table 6.6.1.2.3-1), the fatigue shear resistance for infinite life determined from Eq. 6.6.1.2.5-1 is used for Z_r, with $(\Delta F)_{TH}$ taken as the constant-amplitude fatigue threshold specified for Condition 9.2 in Table 6.6.1.2.3-1. Otherwise, the fatigue shear resistance for finite life determined from Eq. 6.6.1.2.5-2 is used for Z_r, with the detail category constant, *A*, and the fatigue growth constant, *m*, taken as specified for Condition 9.2 in Table 6.6.1.2.3-1. The total number of stress cycles over the fatigue design life, *N*, is computed from Eq. 6.6.1.2.5-3. For a fatigue design life other than 75 years and/or a number of stress cycles per truck passage (*n* from Table 6.6.1.2.5-2) other than 1.0, see the Commentary for Article 6.6.1.2.3.

For more explanation and examples of the determination of the design of shear connectors at the fatigue and strength limit states, see Section 6.3.6.3 of Reference Manual for NHI Course 130122, Design and Evaluation of Steel Bridges for Fatigue and Fracture, Section 6.6.2 of Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures as well as NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge.

The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It performs design calculations addressing the demand on, and resistance of, shear connectors at the fatigue and strength limit states in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.10.3 Special Requirements for Points of Permanent Load Contraflexure

Determination of applicability, *Simple Span Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

Simple Span Bridges:

These provisions are not applicable to simple span bridges, which are only subject to positive flexure.

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

The provisions of this Article are to be used only when shear connectors are omitted in regions of negative flexure, which depends on the preferences of the Owner-agency but is not recommended. The provisions are used to determine the number of additional shear connectors that need to be provided on each side of the points of permanent load contraflexure to develop the fatigue force in the longitudinal reinforcement in the deck due to the negative factored fatigue live load moment at the interior support.

6.10.10.4 Strength Limit State

6.10.10.4.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The provisions of this Article are used to determine the total number of shear connectors required at the strength limit state within specified regions of the span and are applicable to the routine steel I-girder bridges covered by this Guide. The provisions are used to the determine the factored shear resistance of a single shear connector at the strength limit state, Q_r (Eq. 6.10.10.4.1-1), and the minimum number of shear connectors, n (Eq. 6.10.10.4.1-2), that are required over the region of the span under consideration at the strength limit state. For the calculation of the nominal shear resistance, Q_n, used in the determination of Q_r, see the Discussion of Article 6.10.10.4.3 in this Guide. n is determined as the nominal shear force, P, for the region under consideration (see the Discussion of Article 6.10.10.4.2 in this Guide) divided by Q_r.

For more explanation and examples of the determination of the design of shear connectors at the fatigue and strength limit states, see Section 6.3.6.3 of Reference Manual for NHI Course 130122, Design and Evaluation of Steel Bridges for Fatigue and Fracture, Section 6.6.2 of Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures as well as NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge.

The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It performs design calculations addressing the demand on, and resistance of, shear connectors at the fatigue and strength limit states in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.10.4.2 Nominal Shear Force

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

The provisions of this Article are used to compute the nominal shear force, P, at the strength limit state within the region of the span under consideration. P is used in the calculation of the number of studs, n, required within the region under consideration (see the Discussion of Article 6.10.10.4.1 in this Guide). Composite beams with either regularly spaced shear connectors or shear connector clusters have exhibited similar ultimate strength and deflection at service loads in large-scale experimental tests. Only a slight deformation in the concrete and the more heavily stressed connectors are needed to redistribute the horizontal shear to other less heavily stressed connectors.

The number of rows required in the region under consideration at the strength limit state, *NRows*, can be determined as n divided by the number of shear connectors per row. The required pitch, p, in the region at the strength limit state (for comparison with the required pitch, p, determined at the fatigue limit state – see the Discussion of Article 6.10.10.1.2 in this Guide) can then be calculated as the total length of the region (in inches) divided by (*NRows* -1).

The total radial forces in the concrete deck, F_p and F_T, in Eqs. 6.10.10.4.2-1 and 6.10.10.4.2-5, respectively, are not applicable when designing shear connectors for the routine steel I-girder bridges covered by this Guide and may be taken as zero.

For simple span bridges, there is only one region to consider in the calculation of P and n encompassing the entire span. Use Eq. 6.10.10.4.2-1 to calculate P.

For multi-span continuous bridges (rolled beam and plate girder bridges), there are two regions of each span to consider in the calculation of P and n. For continuous spans with shear connectors provided along the entire span length, which is recommended for the routine steel I-girder bridges covered by this Guide, the regions are as follows:

- For end spans:
    - Region between the point of maximum *live load* moment and the end support - use Eq. 6.10.10.4.2-1 to calculate P.

- Region between point of maximum *live load* moment and the adjacent interior support – use Eq. 6.10.10.4.2-5 to calculate *P*.
- For interior spans:
  - Region between the point of maximum *live load* moment and the left interior support - use Eq. 6.10.10.4.2-5 to calculate *P*.
  - Region between point of maximum *live load* moment and the right interior support – use Eq. 6.10.10.4.2-5 to calculate *P*.

For continuous spans with shear connectors omitted in regions of negative flexure, which depends on the preferences of the Owner-agency but is not recommended, the regions are as follows:

- For end spans:
  - Region between the point of maximum *live load* moment and the end support - use Eq. 6.10.10.4.2-1 to calculate *P*.
  - Region between the point of maximum *live load* moment and the adjacent point of steel dead load contraflexure – use Eq. 6.10.10.4.2-1 to calculate *P*.
- For interior spans:
  - Region between the point of maximum *live load* moment and the left point of steel dead load contraflexure - use Eq. 6.10.10.4.2-1 to calculate *P*.
  - Region between point of maximum *live load* moment and the right point of steel dead load contraflexure – use Eq. 6.10.10.4.2-1 to calculate *P*.

For more explanation and examples of the determination of the design of shear connectors at the fatigue and strength limit states, see Section 6.3.6.3 of Reference Manual for NHI Course 130122, Design and Evaluation of Steel Bridges for Fatigue and Fracture, Section 6.6.2 of Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures as well as NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge.

The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It performs design calculations addressing the demand on, and resistance of, shear connectors at the fatigue and strength limit states in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.10.4.3 Nominal Shear Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

The provisions of this Article are used to determine the nominal shear resistance, Q_n, of a single shear connector at the strength limit state. The 0.70 factor in Eq. 6.10.10.4.3-1 accounts for a shear-to-tensile strength factor found in the analysis along with an adjustment to experimental database values to attain a target reliability. The analysis showed that if a shear-to-tensile strength factor of 0.70 was used, then using a resistance factor for shear connectors, ϕ_{sc} = 1.0 (Article 6.5.4.2), is appropriate. The minimum tensile strength of the shear connector, F_u, is 60 ksi (Article 6.4.4).

See the Discussion of Article 6.10.10.4.2 in this Guide for further explanation of how to determine the demand on shear connectors at the strength limit state.

6.10.11 Web Stiffeners

6.10.11.1 Web Transverse Stiffeners

6.10.11.1.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article provides general design requirements for web transverse stiffeners. Web transverse stiffeners on routine plate-girder bridges are typically plates welded to only one side of the web, except for cross-frame or diaphragm connection plates on interior girders, which are typically placed on both sides of the web. Intermediate web transverse stiffeners should be kept the same size along the length of the girder; avoiding multiple plate sizes facilitates the use of repetitive manufacturing techniques and reduces the possibility of placement errors. The minimum thickness used for web stiffeners and connection plates should be ½ inch to facilitate welding (see discussion of stiffener welding in Section 9.2.5 of the FHWA Bridge Welding Reference Manual). Section C1.3 of the AASHTO-NSBA Steel Bridge Collaboration Guideline G12.1-2020 Guidelines to Design for Constructability and Fabrication provides recommended dimensions for these members that will allow fabricators to use either steel plate material or flat steel bar stock for stiffeners and connection plates.

For the routine steel rolled-beam bridges covered by this Guide, web transverse stiffeners are typically not needed, except for use as cross-frame or diaphragm connection plates which do not serve as web transverse stiffeners for shear. A possible exception to the requirement in the 3rd paragraph of this Article for connection plates on rolled-beam bridges is provided in Article 6.6.1.3.1 (see the Discussion of Article 6.6.1.3.1 in this Guide).

The provisions related to web transverse stiffeners on horizontally curved girders and longitudinally stiffened web panels are not applicable to the routine steel I-girder bridges covered by this Guide.

For further information on the design of web transverse stiffeners, consult Section 6.6.6.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructureshttps://www.fhwa.dot.gov/bridge/pubs/nhi15047.pdf. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

For helpful practical guidance on the design and detailing of web transverse stiffeners for economical fabrication, consult AASHTO-NSBA Steel Bridge Collaboration Guidelines G12.1-2020 Guidelines to Design for Constructability and Fabrication and G1.4-2006 Guidelines for Design Details.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design loads and resulting stresses, and the corresponding resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available. Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

6.10.11.1.2 Projecting Width

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article provides two projecting width requirements for web transverse stiffeners that set limits for the stiffener width and thickness in terms of the web depth and flange width. b_f in Eq. 6.10.11.1.2-2 is to be taken as the full width of the widest compression flange within the field section under consideration; the requirement for tub girders in the computation of b_f is not applicable. For the routine rolled steel beam bridges covered by this Guide, the requirements of this Article apply only to the design of the cross-frame or diaphragm connection plates which do not serve as web transverse stiffeners for shear, because rolled beams typically do not require transverse stiffeners to meet shear design requirements for their webs.

For further information on the design of web transverse stiffeners, consult Section 6.6.6.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

For helpful practical guidance on the design and detailing of web transverse stiffeners for economical fabrication, consult AASHTO-NSBA Steel Bridge Collaboration Guidelines G12.1-2020 Guidelines to Design for Constructability and Fabrication and G1.4-2006 Guidelines for Design Details.

6.10.11.1.3 Moment of Inertia

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article contains requirements for the minimum required moment of inertia, I_t, of a web transverse stiffener, where I_t is taken about the edge in contact with the web for single stiffeners and about the mid-thickness of the web for stiffener pairs. This provides the web transverse stiffener with sufficient rigidity to maintain a vertical line of near zero lateral deflection of the web along the line of the stiffener so that the web can adequately develop the shear-buckling resistance or the combined shear-buckling and post-buckling tension-field resistance, as applicable (see the Discussion of Article 6.10.9 in this Guide).

There are two general conditions, each with a different set of equations, for determining I_t. The first condition is for web transverse stiffeners adjacent to web panels in which neither panel is subject to post-buckling tension-field action, e.g., unstiffened web panels adjacent to each other or adjacent to an end panel (Eqs. 6.10.11.1.3-1 and 6.10.11.1.3-2). The provisions clarify for this condition to apply, neither web panel adjacent to the stiffener is subjected to a factored shear force, V_u, larger than the factored shear resistance, $\phi_v V_{cr}$; in other words, in both web panels adjacent to the stiffener the factored shear demand must be less than the factored shear-yielding or shear-buckling resistance, which is defined in Article 6.10.9. In this case, the moment of inertia requirement is taken as the smaller of the two limits given by Eqs. 6.10.11.1.3-1 or 6.10.11.1.3-2. This condition would typically apply for the design of cross-frame or diaphragm connection plates on rolled-beam bridges, which do not serve as web transverse stiffeners for shear.

The second condition is for web transverse stiffeners adjacent to web panels subject to post-buckling tension-field action, e.g., stiffened interior web panels (Eqs. 6.10.11.1.3-7 and 6.10.11.1.3-8). The provisions clarify that this condition applies when either one or both web panels adjacent to the stiffener is subjected to a factored shear force, V_u, larger than the factored shear resistance, $\phi_v V_{cr}$; in other words, this condition applies if the factored shear demand in one or both web panels adjacent to the stiffener is greater than the factored shear-yielding or shear-buckling resistance, which is defined in Article 6.10.9, such that the web may be subject to post-buckling tension-field action. Note that the moment of inertia requirement given by Eq. 6.10.11.1.3-7 depends on the ratio ρ_w. The value to use for ρ_w depends on whether both web panels adjacent to the transverse stiffener are subject to post-buckling tension-field action, in which case the equation given in the first bulleted item underneath Eq. 6.10.11.1.3-8 is used to compute ρ_w. Otherwise, the equation given in the second bulleted item is used.

The Article further clarifies that the calculation of the ratio of the shear-buckling resistance to the shear-yield strength, *C*, (as determined by Eq. 6.10.9.3.2-4, 6.10.9.3.2-5, or 6.10.9.3.2-6, as applicable) used in determining the required moment of inertia of the stiffener, is to be based on a value of the shear buckling coefficient, *k*, that is consistent with the character of the web as being unstiffened or stiffened (i.e., consistent with the value of the shear buckling coefficient, *k*, used to calculate the factored shear resistance, $\phi_v V_{cr}$, at the location being investigated).

Eq. 6.10.11.1.3-11 provides a minimum moment of inertia requirement for transverse stiffeners used in web panels with longitudinal stiffeners. This requirement is not applicable for the routine steel I-girder bridges covered by this Guide, which do not contain web longitudinal stiffeners.

For further information on the design of web transverse stiffeners, consult Section 6.6.6.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

For helpful practical guidance on the design and detailing of web transverse stiffeners for economical fabrication, consult AASHTO-NSBA Steel Bridge Collaboration Guidelines G12.1-2020 Guidelines to Design for Constructability and Fabrication and G1.4-2006 Guidelines for Design Details.

6.10.11.2 Bearing Stiffeners

6.10.11.2.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article provides general design requirements for bearing stiffeners. Plates welded to both sides of the web are typically used for bearing stiffeners in the routine I-girder bridges covered by this Guide. The stiffeners must extend the full depth of the web. In plate-girder bridges, bearing stiffeners must be provided at support locations. For rolled-beam bridges, the provisions of Article D6.5 related to the limit states of web local yielding and web crippling must be checked to determine if bearing stiffeners are necessary at support locations or if they can be omitted (see the Discussion of Article D6.5 in this Guide).

The 10th Edition of the AASHTO LRFD BDS also introduced provisions and commentary related to the use of half-round I-girder bearing stiffeners at supports. Half-round bearing stiffeners (also sometimes referred to as “split pipe bearing stiffeners”) are comprised of half of a round, pipe-shaped section on each side of the girder web. Half-round bearing stiffeners can be fabricated by splitting a pipe in half or by bending a flat plate to a specified radius. Each half-round section is attached to the girder web and flanges using fillet welds, which are also intended to seal the section to prevent exposure of the inside of the half-round section to air or water. A connection plate is fillet welded to the half-round bearing stiffener without attachment of the connection plate to either of the girder’s flanges; the connection plate is oriented parallel to the skew and radial to the center of the half-round section. The use of half-round bearing stiffeners allows for a perpendicular attachment of the cross-frame or diaphragm connection plate, avoiding the need to attach the connection plate using a severely skewed welded connection (which may be impractical to fabricate) and eliminating the need for a bent gusset plate. The Commentary of Article 6.10.8.2.3 (see the Discussion of Article 6.10.8.2.3) suggests that the use of half-round I-girder bearing stiffeners can also provide increased torsional warping restraint, resulting in a beneficial increase in lateral-torsional buckling resistance locally in the unbraced length containing this type of stiffener, and provides simplified guidance for quantifying this beneficial effect.

The use of half-round I-girder bearing stiffeners is typically reserved for cases of fairly severe skew (i.e., greater than 20 degrees); consequently, their use is not recommended for the routine steel I-girder bridges covered by this Guide (which, by definition, feature little or no skew). However, their use may be considered on some moderately skewed steel I-girders bridges when appropriate.

The provisions of Article D6.5 are also used to determine if bearing stiffeners are necessary at other locations in plate-girder or rolled-beam bridges subjected to concentrated loads, where the loads are not transmitted through a deck or deck system. A common example of this situation is a jacking point on a girder or on a diaphragm at an end and/or interior support, where a jack may be placed under the girder or diaphragm and used to lift the superstructure to facilitate bearing replacement or other maintenance activities.

Bearing stiffeners serving as connection plates for cross-frames or diaphragms must be attached to both flanges of the cross-section. The use of fillet welds to attach the stiffeners to the flanges is recommended. The use of complete joint penetration groove welds to attach the stiffeners to the flange through which it receives its load is permitted but is discouraged to significantly reduce the welding deformation of the flange associated with large, complete joint penetration groove welds. The method of attachment that is used is dependent on the preferences of the Owner-agency.

In particularly long bridges supported by bearings with sliding surfaces, it may be necessary to consider providing so-called “auxiliary bearing stiffeners.” This is done in situations where the longitudinal movement of the girder under thermal expansion or contraction is sufficiently large enough that the bearing stiffener may not be located above the bearing at the limits of the design temperature range; in these cases, additional bearing stiffeners are sometimes provided. Care should be taken to detail the location and size of the primary and auxiliary bearing stiffeners such that there is sufficient access for welding to attach the stiffeners to the girder flanges and web (see the Discussion of Article 6.10.11.2.4b in this Guide). Consideration should also be given to the specific finishing and welding details; the flange may undergo deformations when the first stiffener is welded such that a “mill to bear” or “finish to bear” condition may be difficult to achieve in the adjacent stiffener. Calling out complete joint penetration groove welds in conjunction with "mill to bear" or "finish to bear" requirements is discouraged.

For further information on the design of bearing stiffeners, consult Section 6.6.6.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

For helpful practical guidance on the design and detailing of bearing stiffeners for economical fabrication, consult AASHTO-NSBA Steel Bridge Collaboration Guidelines G12.1-2020 Guidelines to Design for Constructability and Fabrication and G1.4-2006 Guidelines for Design Details. Section 9.3.2 of the FHWA Bridge Welding Reference Manual also has good guidance on detailing of bearing and jacking stiffeners.

6.10.11.2.2 Minimum Thickness

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Partially applicable.

Discussion:

This Article specifies the required minimum thickness for bearing stiffeners.

For flat plate bearing stiffeners, the required minimum thickness, t_p, is expressed in terms of the projecting stiffener width, b_t, and the specified minimum yield strength of the stiffeners, F_{ys} (Eq. 6.10.11.2.2-1). This requirement is intended to prevent local buckling of the stiffener plates. The projecting width of the bearing stiffeners should extend as closely as practical to the outer edges of the flanges. This requirement is applicable for bearing stiffeners used in the routine plate-girder bridges covered by this Guide and is only applicable for routine rolled-beam bridges if bearing stiffeners are necessary at support locations or elsewhere.

For half-round bearing stiffeners, the required minimum thickness, t, is expressed as a function of the outside radius of the half-round section, r, and the specified minimum yield strength of the stiffeners, F_{ys} (Eq. 6.10.11.2.2-2). This requirement is not applicable to the routine steel I-girder bridges covered by this Guide, since the use of half-round bearing is typically reserved for cases of fairly severe skew; consequently, their use is not recommended for the routine steel I-girder bridges covered by this Guide (which, by definition, feature little or no skew). However, their use may be considered on some moderately skewed steel I-girders bridges when appropriate. See the Discussion of Article 6.10.11.2 in this Guide for more information on half-round I-girder bearing stiffeners.

For further information on the design of bearing stiffeners, consult Section 6.6.6.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

For helpful practical guidance on the design and detailing of bearing stiffeners for economical fabrication, consult AASHTO-NSBA Steel Bridge Collaboration Guidelines G12.1-2020 Guidelines to Design for Constructability and Fabrication and G1.4-2006 Guidelines for Design Details.

6.10.11.2.3 Bearing Resistance

Determination of applicability, *Simple Span Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Applicable.

Discussion:

This Article specifies a bearing resistance requirement for bearing stiffeners (Eq. 6.10.11.2.3-1). The factored bearing resistance given by Eq. 6.10.11.2.3-1 must equal or exceed the factored bearing reaction at the strength limit state. Each stiffener should be finished to bear against the flange through which it receives its load. Note that the bearing stiffener area to be used in this check is not the gross stiffener area. Rather, it is the bearing area, A_{pn}, which excludes the portions of the stiffeners that must be clipped to facilitate the web-to-flange fillet weld and any of the stiffener area extending beyond the edges of the flange.

This requirement is applicable for bearing stiffeners used in the routine plate-girder bridges covered by this Guide and is only applicable for routine rolled-beam bridges if bearing stiffeners are necessary at support locations or elsewhere.

For further information on the design of bearing stiffeners, consult Section 6.6.6.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the

AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

For helpful practical guidance on the design and detailing of bearing stiffeners for economical fabrication, consult AASHTO-NSBA Steel Bridge Collaboration Guidelines G12.1-2020 Guidelines to Design for Constructability and Fabrication and G1.4-2006 Guidelines for Design Details.

6.10.11.2.4 Axial Resistance of Bearing Stiffeners

6.10.11.2.4a General

Determination of applicability, *Simple Span Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Applicable.

Discussion:

The provisions of this Article are used to determine the factored axial resistance of an effective column section consisting of the bearing stiffeners and a portion of the web (see the Discussion of Article 6.10.11.2.4b in this Guide), which may be included when welded stiffeners are used. The factored axial resistance is determined according to the provisions of Articles 6.9.2.1 and 6.9.4.1.1 using the specified minimum yield strength of the stiffener plates, F_{ys} (see the Discussion of Articles 6.9.2.1 and 6.9.4.1.1 in this Guide). The effective length of the column is taken as 0.75 times the web depth, which assumes some level of fixity of each end of the stiffener plates. The radius of gyration of the column is taken about the mid-thickness of the web. The factored axial resistance of the effective column section must equal or exceed the factored bearing reaction at the strength limit state.

This requirement is applicable for bearing stiffeners used in the routine plate-girder bridges covered by this Guide and is only applicable for routine rolled-beam bridges if bearing stiffeners are necessary at support locations or elsewhere.

For further information on the design of bearing stiffeners, consult Section 6.6.6.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

For helpful practical guidance on the design and detailing of bearing stiffeners for economical fabrication, consult AASHTO-NSBA Steel Bridge Collaboration Guidelines G12.1-2020

Guidelines to Design for Constructability and Fabrication and G1.4-2006 Guidelines for Design Details.

6.10.11.2.4b Effective Section

Determination of applicability, *Simple Span Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Partially applicable.

Discussion:

The provisions of this Article are used to determine the effective column section of plate bearing stiffeners including a portion of the web for computing the factored axial resistance in Article 6.10.11.2.4a (see the Discussion of Article 6.10.11.2.4a in this Guide). For stiffeners consisting of two plates welded to the web, the effective column section is to consist of the two stiffener elements, plus a centrally located strip of web extending not more than $9t_w$ on each side of the stiffeners. If more than one pair of stiffeners is used, the effective column section is to consist of the stiffener elements, plus a centrally located strip of web extending not more than $9t_w$ on each side of the outer projecting elements of the group. If more than one pair of stiffeners is used, it is recommended that a minimum spacing of 8.0 inches of 1.5 times the stiffener width be provided between the stiffeners for welding access. The maximum spacing between the stiffeners should not exceed $1.09t_w\sqrt{E/F_{ys}}$ to prevent an effective width reduction on the web of the section between the stiffeners, where t_w is the thickness of the web. For half-round bearing stiffeners, the effective column section is to be taken as a closed section consisting of the stiffener elements only. This requirement is not applicable to the routine steel I-girder bridges covered by this Guide, since the use of half-round bearing is typically reserved for cases of fairly severe skew; consequently, their use is not recommended for the routine steel I-girder bridges covered by this Guide (which, by definition, feature little or no skew). However, their use may be considered on some moderately skewed steel I-girders bridges when appropriate. See the Discussion of Article 6.10.11.2 in this Guide for more information on half-round I-girder bearing stiffeners.

These provisions are applicable for bearing stiffeners used in the routine plate-girder bridges covered by this Guide and is only applicable for routine rolled-beam bridges if bearing stiffeners are necessary at support locations or elsewhere. The provisions specified for stiffeners bolted to the web and for bearing stiffeners at interior supports on continuous-span hybrid members do not apply to the routine steel I-girder bridges covered by this Guide.

For further information on the design of bearing stiffeners, consult Section 6.6.6.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge, NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the

AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

For helpful practical guidance on the design and detailing of bearing stiffeners for economical fabrication, consult AASHTO-NSBA Steel Bridge Collaboration Guidelines G12.1-2020 Guidelines to Design for Constructability and Fabrication and G1.4-2006 Guidelines for Design Details.

6.10.11.3 Web Longitudinal Stiffeners

6.10.11.3.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article cover the general provisions related to the design of web longitudinal stiffeners. The routine steel I-girder bridges covered by this Guide do not have web longitudinal stiffeners and therefore these provisions are not applicable.

6.10.11.3.2 Projecting Width

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article cover the specific design provisions related to the projecting width of web longitudinal stiffeners. The routine steel I-girder bridges covered by this Guide do not have web longitudinal stiffeners and therefore these provisions are not applicable.

6.10.11.3.3 Moment of Inertia and Radius of Gyration

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article cover the specific design provisions related to the moment of inertia and radius of gyration of web longitudinal stiffeners. The routine steel I-girder bridges covered by this Guide do not have web longitudinal stiffeners and therefore these provisions are not applicable.

6.10.12 Cover Plates

6.10.12.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article cover the general provisions related to the design of cover plates for I-section flexural members. The routine steel I-girder bridges covered by this Guide do not have cover plates and therefore these provisions are not applicable.

6.10.12.2 End Requirements

6.10.12.2.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article cover the general design provisions related to the end requirements for cover plates for I-section flexural members. The routine steel I-girder bridges covered by this Guide do not have cover plates and therefore these provisions are not applicable.

6.10.12.2.2 Welded Ends

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article cover the design provisions specific to welded ends of cover plates for I-section flexural members. The routine steel I-girder bridges covered by this Guide do not have cover plates and therefore these provisions are not applicable.

6.10.12.2.3 Bolted Ends

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article cover the design provisions specific to bolted ends of cover plates for I-section flexural members. The routine steel I-girder bridges covered by this Guide do not have cover plates and therefore these provisions are not applicable.

6.11 COMPOSITE BOX-SECTION FLEXURAL MEMBERS

6.11.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies general design requirements for composite steel box-section flexural members used in straight or horizontally curved bridges. The provisions are applicable to both composite closed-box and tub-section members. Tub sections have an open top with two separate top flanges laced together with lateral bracing to form a pseudo-box to resist the torsion prior to the hardening of the deck. Tub sections are by far the most commonly used cross-section type for composite box-section flexural members and typically have inclined webs to allow for the use of a narrower and more economical bottom flange plate while enjoying the advantage of a wider spacing of the webs supporting the deck. Closed-box sections enclosed at the top with a steel plate that is composite with the concrete deck are rarely, if ever, used for these members since OSHA regulations make it very expensive and impractical to work inside a closed box. Box-girder cross-sections can consist of multiple single-cell steel boxes (most common), one single-cell steel box, or a single multi-cell steel box. The latter type is rarely employed and is not covered in the AASHTO LRFD BDS.

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

For further information on the design of composite steel box-section flexural members, consult the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples, consult the NSBA's Steel Bridge Design Handbook – Design Example 4: Three-Span Continuous Straight Composite Steel Tub-Girder Bridge and NSBA's Steel Bridge Design Handbook – Design Example 5: Three-Span Continuous Horizontally Curved Composite Steel Tub-Girder Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.11.1.1 Stress Determinations

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This article contains provisions related to the effective width of a box flange in a composite box-section member subject to flexure (see the Discussion of Article 6.11.1 in this Guide) to account for the effects of shear lag in the calculation of flexural stresses in the box section, where a box flange is defined in the AASHTO LRFD BDS as a flange that is connected to two webs. This article also contains provisions related to the stress determinations in a composite box-section flexural member, including the determination of the live-load distribution to the individual boxes in the cross-section, the section of an exterior girder assumed to resist the factored wind loading, the types of box sections for which St. Venant torsional shear stresses and transverse bending and longitudinal warping stresses due to cross-section distortion must be considered, and a specified limit on the factored torsional shear stress in such boxes at the strength limit state.

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.1.2 Bearings

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article contains provisions related to the use of a single or double bearing arrangement to support a composite box-section flexural member (see the Discussion of Article 6.11.1 in this Guide). The potential use of tie-down bearings is also discussed.

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.1.3 Flange-to-Web Connections

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article contains provisions related to the size and placement of welded flange-to-web connections in a composite box-section flexural member (see the Discussion of Article 6.11.1 in this Guide), and the minimum number of intermediate internal cross-frames or diaphragms that must be provided within each span to use fillet welds for these connections.

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.1.4 Access and Drainage

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article covers the provision of access holes in the bottom flange of composite box-section flexural members (see the Discussion of Article 6.11.1 in this Guide) for inspection. Provisions are specified for the placement of the holes, the need for reinforcement of the holes, and the checking of local buckling of the remaining flange on each side of the hole at access holes in bottom flanges subject to compression. Ventilation and drainage of the interior of the box section is also discussed.

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.2 Cross-Section Proportion Limits

6.11.2.1 Web Proportions

6.11.2.1.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies general design requirements for webs of composite box-section flexural members (see the Discussion of Article 6.11.1 in this Guide), including the limiting slope of inclined webs.

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.2.1.2 Webs without Longitudinal Stiffeners

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies the limiting web slenderness for webs of composite box-section flexural members (see the Discussion of Article 6.11.1 in this Guide) without longitudinal stiffeners

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.2.1.3 Webs with Longitudinal Stiffeners

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies the limiting web slenderness for webs of composite box-section flexural members (see the Discussion of Article 6.11.1 in this Guide) with longitudinal stiffeners

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.2.2 Flange Proportions

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies the flange proportioning requirements for top flanges of composite tub-section flexural members subject to tension or compression (see the Discussion of Article 6.11.1 in this Guide). Note that Eq. 6.11.2.2-3 is only to apply to top flanges of built-up tub-section members and need not be applied to tub-section members that are fabricated from a single plate. This article also specifies the flange proportioning requirements for longitudinally unstiffened and longitudinally stiffened box flanges subject to tension or compression, as well as the proportioning requirements for flange extensions on box flanges.

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.2.3 Special Restrictions on Use of Live Load Distribution Factor for Multiple Box Sections

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies restrictions for straight bridges utilizing multiple composite box-section flexural members (see the Discussion of Article 6.11.1 in this Guide) that must be met in order to employ the lateral live-load distribution factor given in Article 4.6.2.2.2b for straight multiple steel box sections (see the Discussion of Article 4.6.2.2.2b in this Guide). Otherwise, a refined analysis must be used to determine the live-load distribution. Furthermore, for bridges satisfying these restrictions and with an effective box-flange width not exceeding one-fifth of the effective span defined in Article 6.11.1.1 (see the Discussion of Article 6.11.1.1 in this Guide), shear due to St. Venant torsion and secondary distortional bending stress effects may be neglected.

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.3 Constructibility

6.11.3.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article refers to Article 6.10.3 for the general provisions related to the constructibility design of composite steel box-section flexural members (see the Discussion of Article 6.10.3 in this Guide). The provisions also require that the need for bracing to maintain individual box-section geometry throughout all stages of construction be investigated.

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.3.2 Flexure

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article refers to the provisions of Articles 6.10.3.2.1 through 6.10.3.2.3 for the checking of the top flanges of composite steel tub-section flexural members for constructibility (see the Discussion of Article 6.10.3.2.1 in this Guide) and defines the unbraced lengths for computing top-flange lateral bending moments for such flanges when braced by a full-length or partial-length top lateral bracing system. Lateral-torsional buckling between panel points need not be checked for top flanges of tub girders with a full-length or partial-length top lateral bracing system and also need not be checked between the transition points from the laterally braced to the unbraced sections when a partial-length top lateral bracing system is used. For tub girders with no top lateral bracing system, or with top lateral bracing systems that do not extend at least between 20 to 25 percent of the span length from each girder end support, an elastic buckling analysis using a three-dimensional shell-element model that captures the significant aspects of the nonprismatic geometry must be conducted to investigate the global LTB stability of each span of an individual noncomposite tub girder for the assumed deck placement sequence.

Provisions are also provided to check noncomposite box flanges and continuously braced box flanges subject to compression or tension during construction and also to check composite box flanges before the concrete deck has hardened or is made composite, where a box flange is defined in the AASHTO LRFD BDS as a flange that is connected to two webs. The resistance equations for box flanges in this Article include the consideration of the St. Venant torsional shear stress in the flange due to the torque applied to the noncomposite section for the specific cases in which the torsional shear must be considered (see the Discussion of Articles 6.11.1.1 and 6.11.2.3 in this

Guide). Since post-buckling resistance is assumed at the strength limit state in computing the nominal flexural resistance of noncomposite box flanges subject to compression in which the box flange is longitudinally unstiffened and not classified as compact (see the Discussion of Article 6.11.8.2.2b in this Guide), or the box flange is longitudinally stiffened and contains slender longitudinally stiffened plate panels as defined in Article E6.1.2 (see the Discussion of Articles 6.11.8.2.3 and E6.1.2 in this Guide), the flange in such cases must also satisfy Eq. 6.11.3.2-1 to ensure that plate local buckling due to flexural stresses does not occur theoretically during construction. Flange lateral bending is not a consideration for box flanges. St. Venant torsional shears are typically neglected in the top flanges of tub sections.

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.3.3 Shear

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article indicates that when checking the shear requirement specified in Article 6.10.3.3 for composite steel box-section flexural members during construction (see the Discussion of Article 6.10.3.3 in this Guide), the provisions of Article 6.11.9 also apply (see the Discussion of Article 6.11.9 in this Guide).

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.4 Service Limit State

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article refers to Article 6.10.4 for the checking of the service limit state design provisions for composite steel box-section flexural members (see the Discussion of Article 6.10.4 in this Guide), with certain exceptions as listed. Since post-buckling resistance is assumed at the strength limit state in computing the nominal flexural resistance of noncomposite box flanges subject to compression in which the box flange is longitudinally unstiffened and is not classified as compact (see the Discussion of Article 6.11.8.2.2b in this Guide), or the box flange is longitudinally stiffened and contains slender longitudinally stiffened plate panels as defined in Article E6.1.2 (see the Discussion of Articles 6.11.8.2.3 and E6.1.2 in this Guide), the flange in such cases must also satisfy Eq. 6.11.4-1 to ensure that plate local buckling due to flexural stresses does not occur theoretically at the service limit state. This Article also indicates the optional service limit state moment redistribution procedures given in Appendix B6 are not to be applied to composite box-section flexural members (see the Discussion of Appendix B6 in this Guide) because the applicability of these provisions to box sections has not been demonstrated. The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.5 Fatigue and Fracture Limit State

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article refers to Article 6.10.5 for the checking of the fatigue limit state design provisions for composite steel box-section flexural members (see the Discussion of Article 6.10.5 in this Guide). Specific requirements related to the checking of fatigue of shear connectors and the special fatigue requirement for webs given in Article 6.10.5.3 (see the Discussion of Article 6.10.5.3 in this Guide) for such members are also provided. Situations for which longitudinal warping stresses and transverse bending stresses due to cross-section distortion may be of concern for fatigue are also discussed along with suggested approaches to calculate and control these stresses where necessary. This Article further indicates that in the calculation of dead load and live load stresses and live load stress ranges for fatigue design, the box-flange area of the gross cross-section is to be reduced, if applicable, to account for shear lag as specified in Article 6.11.1.1 (see the Discussion of Article 6.11.1.1 in this Guide).

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The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.6 Strength Limit State

6.11.6.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article refers to the applicable Strength load combinations given in Table 3.4.1-1, which are utilized in the design checks at the strength limit state for composite steel box-section flexural members.

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.6.2 Flexure

6.11.6.2.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article refers to the provisions Article 6.10.1.8 if there are holes in the tension flange of a composite steel box-section flexural member; e.g., at a bolted splice (see the Discussion of Article 6.10.1.8 in this Guide).

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.6.2.2 Sections in Positive Flexure

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article defines the requirements for a composite box section in regions of positive flexure to qualify as a compact or a noncompact section at the strength limit state, and where the provisions to design each type of section are located in Article 6.11.7 (see the Discussion of Article 6.11.7 in this Guide). This Article also refers to the ductility requirement given in Article 6.10.7.3 to provide a ductile mode of failure, which must be checked for both compact and noncompact sections (see the Discussion of Article 6.10.7.3 in this Guide).

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.6.2.3 Sections in Negative Flexure

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article indicates that the provisions of Article 6.11.8 are to apply for a composite box section in regions of negative flexure at the strength limit state (see the Discussion of Article 6.11.8 in this Guide). This Article also indicates the optional strength limit state moment redistribution procedures given in Appendix B6 are not to be applied to composite box-section flexural members (see the Discussion of Appendix B6 in this Guide) because the applicability of these provisions to box sections has not been demonstrated. For sections in negative flexure, the provisions of Article 6.11.8 limit the nominal flexural resistance to be less than or equal to the moment at first yield for all types of composite box-girder bridges. As a result, all sections in negative flexure are conservatively treated as slender-web sections at the strength limit state regardless of their web slenderness, and the nominal flexural resistance for these sections is conveniently expressed in terms of the elastically computed flange stress on the gross cross-section. The box-flange areas of the gross cross-section are to be reduced, if applicable, to account for shear lag as specified in Article 6.11.1.1 in the calculation of the stress (see the Discussion of Article 6.11.1.1 in this Guide).

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.6.3 Shear

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article simply points to the provisions of Article 6.11.9 for determining the factored shear resistance of a composite box-section flexural member at the strength limit state (see the Discussion of Article 6.11.9 in this Guide).

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.6.4 Shear Connectors

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article simply points to the provisions of Article 6.10.10.4 for determining the factored shear resistance of shear connectors for composite box-section flexural members at the strength limit state (see the Discussion of Article 6.10.10.4 in this Guide). These provisions further refer to the provisions of Article 6.11.10, as applicable (see the Discussion of Article 6.11.10 in this Guide).

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.7 Flexural Resistance—Sections in Positive Flexure

6.11.7.1 Compact Sections

6.11.7.1.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides the relationship that must be satisfied at the strength limit state for compact composite box sections in regions of positive flexure. Most composite sections in regions of positive flexure in straight steel box girder bridges without holes in the tension flange will qualify as compact sections. Sections that qualify as compact sections may conservatively be treated as noncompact sections (see the Discussion of Article 6.11.7.2 in this Guide), if desired.

For compact sections, the nominal flexural resistance is permitted to exceed the moment at first yield assuming there are no holes in the tension flange at the section under consideration. The moment at first yield, M_y, is defined as the moment at which an outer fiber first attains the yield stress (see the Discussion of Article D6.2.2 in this Guide). The nominal flexural resistance is not permitted to exceed the plastic moment, M_p. M_p is defined as the resisting moment of a fully yielded cross-section (see the Discussion of Article D6.1 in this Guide). For compact sections, the nominal flexural resistance is expressed in terms of moment for reasons discussed in the Commentary for Article 6.10.6.1.

Flange lateral bending is not a consideration for top flanges because the flanges are continuously braced by the concrete deck. Flange lateral bending is also not a consideration for bottom box flanges, where a box flange is defined in the AASHTO LRFD BDS as a flange that is connected to two webs.

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.7.1.2 Nominal Flexural Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article refers to the provisions of Article 6.10.7.1.2 for determining the nominal flexural resistance, M_n, of compact composite box sections in regions of positive flexure at the strength limit state (see the Discussion of Article 6.10.7.1.2 in this Guide). The single exception is that for continuous spans, M_n must always be subject to the limitation of $1.3R_hM_y$ given in Eq. 6.10.7.1-2-3, as the provisions of Appendix B6 described in the two bulleted items in Article 6.10.7.1.2 are not applicable to box sections. The reasons for this limitation are described further in the Discussion of Article 6.10.7.1.2 in this Guide.

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.7.2 Noncompact Sections

6.11.7.2.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides the relationships that must be satisfied at the strength limit state for the compression and tension flanges of noncompact composite box sections in regions of positive flexure. Composite sections in regions of positive flexure in horizontally curved steel box girder bridges must be treated as noncompact sections. For noncompact sections (and for compact sections with holes in the tension flange), the nominal flexural resistance is not to exceed the moment at first yield. The moment at first yield, M_y, is defined as the moment at which an outer fiber first attains the yield stress (see the Discussion of Article D6.2.2 in this Guide). For noncompact sections, the nominal flexural resistance is expressed in terms of the elastically computed flange stress for reasons discussed in the Commentary for Article 6.10.6.1. The box-flange area of the gross cross-section is to be reduced, if applicable, to account for shear lag as specified in Article 6.11.1.1 in the calculation of the total factored stress in each flange (see the Discussion of Article 6.11.1.1 in this Guide).

This Article further limits the maximum total factored longitudinal compressive stress in the concrete deck at the strength limit state for a noncompact box section to $0.6f'_c$ to maintain linear behavior of the concrete, which is assumed in the calculation of the steel flange stresses.

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.7.2.2 Nominal Flexural Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides the equations for computing the nominal flexural resistance of the compression flange, F_{nc}, and the nominal flexural resistance of the tension flange, F_{nt}, for noncompact composite box sections in regions of positive flexure at the strength limit state. For noncompact sections, the elastically computed total factored stress in each flange, determined in accordance with Article 6.10.1.1.1a and Article 6.10.1.1.1b, is compared with the yield stress of the flange times the appropriate flange-stress reduction factors, R_b and R_h (see the Discussion of Articles 6.10.1.1.1a, 6.10.1.1.1b, 6.10.1.10.1 and 6.10.1.10.2 in this Guide). Separate resistance equations are provided for the top compression flanges of tub sections and composite compression flange of closed-box sections and for the bottom tension flanges of tub sections and closed-box sections. The resistance equations for box flanges in this Article include the consideration of the total factored St. Venant torsional shear stress in the flange for the specific cases in which the torsional shear must be considered (see the Discussion of Articles 6.11.1.1 and 6.11.2.3 in this Guide), where a box flange is defined in the AASHTO LRFD BDS as a flange that is connected to two webs. The Commentary for this Article discusses the computation of the flange torsional shear stress.

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.8 Flexural Resistance—Sections in Negative Flexure

6.11.8.1 General

6.11.8.1.1 Box Flanges in Compression

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides the relationship that must be satisfied at the strength limit state for box flanges subject to compression in regions of negative flexure, where a box flange is defined in the AASHTO LRFD BDS as a flange that is connected to two webs. This relationship is intended to check that box flanges subject to compression in composite box-section flexural members will have sufficient strength to resist flange local buckling. Flange lateral bending and lateral-torsional buckling are not a consideration for box flanges in composite box-section members. Equations are also provided in the Commentary for this Article to check the complex combined stress state in bottom box flanges at interior-pier sections in cases where the internal diaphragm shear stresses and/or bending of the internal diaphragm over the bearing sole plate are deemed significant (e.g., boxes supported on single bearings).

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.8.1.2 Continuously Braced Flanges in Tension

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides the relationship that must be satisfied at the strength limit state for continuously braced flanges subjected to tension in regions of negative flexure in composite box-section flexural members. A continuously braced flange is defined as a flange that is encased in concrete or anchored to the concrete deck by shear connectors satisfying the provisions of Article 6.10.10. For continuously braced top flanges of tub sections, lateral flange bending stresses and St. Venant torsional shear stresses are neglected. The torsional shear stresses are not to be neglected in continuously braced box flanges, where a box flange is defined in the AASHTO LRFD BDS as a flange that is connected to two webs.

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.8.2 Flexural Resistance of Noncomposite Box Flanges in Compression

6.11.8.2.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article point to the appropriate articles for determining the nominal flexural resistance, F_{nc}, of box flanges subject to compression both with (see the Discussion of Article 6.11.8.2.2 in this Guide) and without (see the Discussion of Article 6.11.8.2.3 in this Guide) flange longitudinal stiffeners in composite box-section flexural members at the strength limit state, where a box flange is defined in the AASHTO LRFD BDS as a flange that is connected to two webs. Flange lateral bending is not a consideration for box flanges.

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.8.2.2 Longitudinally Unstiffened Flanges

6.11.8.2.2a Classification of Longitudinally Unstiffened Flanges

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides the limiting slenderness, λ_{pf}, for a longitudinally unstiffened box flange in compression to be classified as a compact flange at the strength limit state, where a box flange is defined in the AASHTO LRFD BDS as a flange that is connected to two webs. For longitudinally unstiffened noncomposite box flanges, the flange slenderness, λ_f, is based on the clear flange width between webs, b_{fi}. Longitudinally unstiffened box flanges in compression with a slenderness larger than this limiting slenderness, given by Eq. 6.11.8.2.2a-3, have a compression flange effective width smaller than the gross width of the flange.

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.8.2.2b General Yielding and Compression FLB

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides the equation for calculating the nominal local buckling resistance, F_{nc}, of unstiffened box flanges in composite box-section flexural members subject to compression (i.e., flanges without longitudinal stiffeners) on the gross cross-sectional area based on the combined influence of general yielding and flange local buckling (FLB) at the strength limit state, where a box flange is defined in the AASHTO LRFD BDS as a flange that is connected to two webs.

FLB is considered through the use of an effective cross section, considering the post-buckling response of box flanges subject to compression that are not classified as compact as defined in Article 6.11.8.2.2a (see the Discussion of Article 6.11.8.2.2a in this Guide). Box flanges subject to compression, which are supported by webs along two longitudinal edges, have substantial post-buckling resistance. The influence of the reduced compression-flange effective width on the flexural resistance is addressed by multiplying the plateau strength, $R_bR_hF_{yc}\Delta$, by the ratio of M_{yce} to M_{yc}. M_{yce} is the yield moment with respect to the compression flange determined as specified in Article D6.2.1 or D6.2.3 (see the Discussion of Article D6.2.1 and D6.2.3 in this Guide), as applicable, using the effective width of the compression flange, b_e, calculated as specified in Article 6.9.4.2.2b with F_{cr} taken equal to F_{yc} (see the Discussion of Article 6.9.4.2.2b in this Guide) and including the width of the corner areas and flange extensions. The corresponding compression-flange stresses are calculated on the gross cross-section with a reduction in the gross box-section width to account for shear lag in extreme cases where the box-flange width exceeds one-fifth of the effective span as specified in Article 6.11.1.1 (see the Discussion of Article 6.11.1.1 in this Guide). For sections with a longitudinally unstiffened compact box flange in compression, M_{yce} is equal to M_{yc}, and therefore the ratio of these terms in Eq. 6.11.8.2.2b-1 is equal to 1.0.

The resistance equation includes the consideration of the St. Venant torsional shear stress in the flange for the specific cases in which the torsional shear must be considered (see the Discussion of Articles 6.11.1.1 and 6.11.2.3 in this Guide). The Commentary for this Article discusses the computation of the flange torsional shear stress and the calculation of the flange-stress reduction factors, R_b and R_h, in the resistance equation is described in the "where" list that defines the various terms given in this Article (see also the Discussion of Articles 6.10.1.10.1 and 6.10.1.10.2 in this Guide).

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.8.2.3 Longitudinally Stiffened Flanges

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides the equations for calculating the nominal local buckling resistance, F_{nc}, of longitudinally stiffened box flanges subject to compression in composite box-section flexural members at the strength limit state, where a box flange is defined in the AASHTO LRFD BDS as a flange that is connected to two webs. When a noncomposite longitudinally unstiffened box flange becomes so slender that nominal flexural resistance of the flange decreases to an impractical level, longitudinal stiffeners can be added to the flange. The equation for unstiffened box flanges in compression specified in Article 6.11.8.2.2 is used (see the Discussion of Article 6.11.8.2.2 in this Guide), with the appropriate substitutions made to account for the presence of flange longitudinal stiffeners as listed in this Article.

However, as noted in the Commentary for this Article, given the resistance provided for thin longitudinally unstiffened box flanges by the provisions of Article 6.11.8.2.2b (see the Discussion of Article 6.11.8.2.2b in this Guide), it is apparent that there is essentially no economy to be realized due to longitudinal stiffening in new construction, even for the widest of typical tub girders, unless the constructibility and service flange stress limits on the unstiffened flange specified in Articles 6.11.3.2 and 6.11.4, respectively, are violated (see the Discussion of Articles 6.11.3.2 and 6.11.4 in this Guide). Even in these cases, the more economical route for typical tub girders would likely be to thicken the flange, rather than to provide longitudinal stiffening. Therefore, the provisions of this Article are anticipated to be most useful in the load rating of existing composite box girder bridges with flange longitudinal stiffeners.

Flange longitudinal stiffeners, when utilized, are to satisfy the provisions of Article E6.1.4. Transverse stiffeners, when utilized to strengthen or stiffen a longitudinally stiffened flange, are to satisfy the requirements of Article E6.1.5 (see the Discussion of Articles E6.1.4 and E6.1.5 in this Guide).

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.8.3 Flexural Resistance Based on Tension Flange Yielding

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article are used to compute the nominal flexural resistance, F_{nt}, based on tension flange yielding of continuously braced flanges subjected to tension in regions of negative flexure in composite tub-section flexural members at the strength limit state. For continuously braced flanges subjected to tension in regions of negative flexure in composite closed-box section flexural members, Eq. 6.11.7.2.2-6 is used instead. A continuously braced flange is defined as a flange that is encased in concrete or anchored to the concrete deck by shear connectors satisfying the provisions of Article 6.10.10.

For sections in which $M_{yt} > M_{yc}$, Eq. 6.10.8.3-1 or 6.11.7.2.2-6, as applicable, does not control and tension flange yielding need not be checked, where M_{yc} and M_{yt} are the yield moments with respect to the compression and tension flange, respectively, determined as specified in Article D6.2 (see the Discussion of Article D6.2 in this Guide).

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.9 Shear Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article point to the provisions of Article 6.10.9 for the determination of the factored shear resistance of composite box-section flexural members (see the Discussion of Article 6.10.9 in this Guide), with the exceptions noted below.

For sections with inclined webs, the depth, D, is to be taken as the depth of the web plate measured along the slope and the web is to be designed for the component of the vertical shear in the plane of the inclined web given by Eq. 6.11.9-1. For the specific cases in which the torsional shear must be considered (see the Discussion of Articles 6.11.1.1 and 6.11.2.3 in this Guide), the factored vertical shear, V_u, is to be taken as the sum of the flexural and St. Venant torsional shears. Both webs can be conservatively designed for the critical shear or the shear in the web subject to additive flexural and torsional shear. This Article also specifies the width of box flanges, b_{fc} and b_{ft}, to be used in checking Eq. 6.10.9.3.2-1 (see the Discussion of Article 6.10.9.3.2 in this Guide), where a box flange is defined in the AASHTO LRFD BDS as a flange that is connected to two webs.

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.10 Shear Connectors

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article point to the provisions of Article 6.10.10 for the design of shear connectors for composite box-section flexural members (see the Discussion of Article 6.10.10 in this Guide), with the exceptions noted below. In multi-span continuous box girder bridges, shear connectors must be provided in regions of negative flexure to resist the torsional shear that exists along the entire span in all types of composite box sections.

For the specific cases in which the torsional shear must be considered (see the Discussion of Articles 6.11.1.1 and 6.11.2.3 in this Guide), shear connectors are to be designed for the sum of the flexural and St. Venant torsional shears. For tub sections, the longitudinal fatigue shear range, V_{fat}, is to be computed for the web subject to additive flexural and torsional shear with the resulting pitch also used for the other top flange. The radial fatigue shear range due to curvature, F_{fat1}, is to be ignored for box-section members (see the Discussion of Article 6.10.10.1.2 in this Guide). Provisions for the design of shear connectors at the strength limit state and for the design of shear connectors on composite box flanges in regions of positive flexure in closed-box sections are also provided, where a box flange is defined in the AASHTO LRFD BDS as a flange that is connected to two webs.

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.11.11 Web Stiffeners

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article point to the provisions of Articles 6.10.11.1 and 6.10.11.3 for the design of web transverse stiffeners and web longitudinal stiffeners, respectively, for composite box-section flexural members (see the Discussion of Articles 6.10.11.1 and 6.10.11.3 in this Guide). For the design of bearing stiffeners for such members, the provisions point to the provisions of Article 6.10.11.2 (see the Discussion of Article 6.10.11.2 in this Guide), with the exceptions noted which relate to the fact that bearing stiffeners are typically attached to the internal diaphragms rather than to the inclined webs at supports in these members.

The routine steel I-girder bridges covered by this Guide are not comprised of composite steel box-section flexural members; therefore, the provisions of this Article are not applicable.

6.12 MISCELLANEOUS FLEXURAL MEMBERS

6.12.1 General

6.12.1.1 Scope

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article cover miscellaneous rolled or built-up noncomposite or composite members subject to flexure, often in combination with axial loads; that is, flexural members which are not covered by the provisions of Article 6.10 or 6.11. The nominal flexural resistance of these members is often needed for application in the interaction relationships of Articles 6.8.2.3.1, 6.9.2.2.1, and 6.9.6.3.4, as applicable (see the Discussion of Articles 6.8.2.3.1, 6.9.2.2.1, and 6.9.6.3.4 in this Guide). The specific types of members covered by these provisions are listed in this Article. These types of members are often used in trusses, frames, or arches, or as cross-frame, diaphragm, or lateral bracing members.

Most of the member types discussed in Article 6.12 and its associated sub-Articles are not widely used, or are not used at all, in the routine steel I-girder bridges covered by this Guide. Some are used, but only in limited applications, particularly as cross-frame or diaphragm members.

Tees (WT) and double angles are sometimes be used as cross-frame members, generally only when the axial loads in these members exceed the capacity of single-angle sections. It should be noted that using rolled steel tee (WT) and double-angle sections as cross-frame members for routine steel I-girder bridges is generally discouraged, while the use of single-angle sections is encouraged. The magnitude of the forces in cross-frame members in routine steel I-girder bridges covered by this Guide is generally small enough such that single-angle members are adequate. Cross-frame forces are typically only large enough to warrant the use of rolled steel tee (WT) or double-angle sections in curved and/or skewed steel I-girder bridges. Rolled steel tee (WT) sections are typically quite expensive to fabricate. Tee (WT) sections are cut from full wide-flange (W) shapes and generally require straightening after the cutting process, which adds significant fabrication effort and cost. Double-angle sections are often viewed as problematic from a maintenance perspective; the surfaces between the adjacent angle flanges are difficult or impossible to paint in the field, and/or can suffer from potentially severe pack rust.

Channels may be used as a top chord in an end diaphragm, in which case they need to be designed as a flexural member to support the wheel loads coming onto the end of the deck, or as a diaphragm for a shallow-depth rolled beam structure.

Other member types mentioned in Article 6.12 and its associated sub-Articles could potentially be used in substructures for routine steel I-girder bridges; in those cases, the determination of applicability of the provisions related to those particular members is designated as beyond the scope of superstructure design.

6.12.1.2 Strength Limit State

6.12.1.2.1 Flexure

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article provide the basic relationship that must be satisfied at the strength limit state by the miscellaneous flexural members listed in Article 6.12.1.1 (see the Discussion of Article 6.12.1.1 in this Guide) assuming low or zero levels of axial force in the member and uniaxial flexure. This Article also defines the factored flexural resistance, M_r, of these members to be used in the preceding relationship.

The provisions of Article 6.12.1.2 (and its associated subarticles) are only applicable to miscellaneous flexural members subject to low or zero levels of axial force. As discussed in the Commentary for Article 6.12.1.2.2, for members subject to flexure in combination with a factored concentrically-applied axial force, P_u, in excess of 5 percent of the factored axial resistance of the member, P_r or P_{ry}, as applicable (defined in Articles 6.9.2.2.1 and 6.8.2.3.1, respectively) at the strength limit state, and/or if the member is subject to biaxial bending, the member should instead be checked using the interaction relationships specified in Article 6.8.2.3 or 6.9.2.2, as applicable (see the Discussion of Articles 6.8.2.3 and 6.9.2.2 in this Guide). See the Discussion of Article 6.12.1.2 for more information.

Some of the other member types could potentially be used in the substructure of a routine I-girder bridge and would likely be subject to the interaction relationships, but the determination of applicability of the provisions related to those particular members is designated as beyond the scope of superstructure design.

6.12.1.2.2 Combined Flexure, Axial Load, and Flexural and/or Torsional Shear

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article points to the provisions of Article 6.8.2.3 for combined axial tension, flexure, and flexural and/or torsional shear (see the Discussion of Article 6.8.2.3 in this Guide, or the provisions of Article 6.9.2.2 for combined axial compression, flexure, and flexural and/or torsional shear (see the Discussion of Article 6.9.2.2 in this Guide, when designing a member listed in Article 6.12.1.1 (see the Discussion of Article 6.12.1.1 in this Guide) for the corresponding combined loading conditions. As discussed in the Commentary, the overall provisions of Article 6.12.1.2 and its associated subarticles are specifically applicable only to members subject to uniaxial bending and low or zero axial force (defined as a factored concentrically-applied axial force, P_u, less than 5 percent of the factored axial resistance of the member, P_r or P_{ry}, as applicable, where P_r and P_{ry}, are defined in Articles 6.9.2.2.1 and 6.8.2.3.1, respectively) at the strength limit state. If the member is subject to a higher level of axial force, and/or the member is subject to biaxial bending, the member should instead be checked using the interaction relationships specified in Article 6.8.2.3 or 6.9.2.2, as applicable (see the Discussion of Articles 6.8.2.3 and 6.9.2.2 in this Guide).

See the Discussion of Article 6.12.1.2.1 in this Guide regarding the basic conditions involving axial force and/or flexure for which the interaction relationships given in these articles should be used. For the routine steel I-girder bridges covered by this Guide, the interaction relationships specified in Article 6.9.2.2.1 are likely to be applicable when tees (WT) or double angles are used as cross-frame members, and the members are subject to eccentric axial compression. It should be noted that using rolled steel tee (WT) and double-angle sections as cross-frame members for routine steel I-girder bridges is generally discouraged, while the use of single-angle sections is encouraged. The magnitude of the forces in cross-frame members in routine steel I-girder bridges covered by this Guide is generally small enough such that single-angle members are adequate. Cross-frame forces are typically only large enough to warrant the use of rolled steel tee (WT) or double-angle sections in curved and/or skewed steel I-girder bridges. Rolled steel tee (WT) sections are typically quite expensive to fabricate. Tee (WT) sections are cut from full wide-flange (W) shapes and generally require straightening after the cutting process, which adds significant fabrication effort and cost. Double-angle sections are often viewed as problematic from a maintenance perspective; the surfaces between the adjacent angle flanges are difficult or impossible to paint in the field, and/or can suffer from potentially severe pack rust.

The provisions of this Article may also be applicable to the design of channel sections when they are used as the top chord of end cross-frames or as diaphragms in bridges with shallow-depth beams or girders. Designers are reminded that loads applied to channel sections in a direction parallel to the plane of the web, but offset from the shear center of the section, will induce a torsion in the section. The shear center of a singly-symmetric channel section (such as a typical AISC C or MC channel shape) is generally located at mid-depth of the section, but offset from the web in a direction opposite to the direction in which the flanges are pointed; in other words, the locations of the center of gravity and the shear center are not coincident in a channel shape.

Some of the other member types could potentially be used in the substructure of a routine I-girder bridge and would likely be subject to the interaction relationships, but the provisions related to those particular member types are designated as beyond the scope and not applicable to superstructure design. The last paragraph of this Article related to noncomposite circular tubes, including round HSS (Hollow Structural Sections), under combined loading conditions is not applicable to the routine steel I-girder bridges covered by this Guide as these members are not used in these bridges, unless they are used in the substructure or foundations for the bridge which is considered beyond the scope of superstructure design.

6.12.1.2.3 Flexural Shear and/or Torsion

6.12.1.2.3a General

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article are used to calculate and check the factored shear resistance, V_r, at the strength limit state of the miscellaneous flexural members listed in Article 6.12.1.1 (see the Discussion of Article 6.12.1.1 in this Guide). The provisions are also used to calculate and check the factored torsional resistance, T_r, at the strength limit state of noncomposite circular tubes, including round HSS (Hollow Structural Sections) subject to torsion only or subject to combined flexural shear and torsion. The basic relationships that must be satisfied for each case are provided, along with the definitions of V_r and T_r to be used in these relationships. The Article also points to the appropriate provisions for the calculation of the nominal shear resistance, V_n, and nominal torsional resistance, T_n, as applicable, for each type of member.

For the routine steel I-girder bridges covered by this Guide, the provisions of this Article are only applicable to check the factored shear resistance of a channel if a channel is used as a top chord in an end diaphragm and it is designed as a flexural member to support the wheel loads coming onto the end of the deck, or as a diaphragm for a shallow-depth rolled beam structure. The provisions of Article 6.10.9 would be used to compute V_n in this case (see the Discussion of Article 6.10.9 in this Guide). Designers are reminded that loads applied to channel sections in a direction parallel to the plane of the web, but offset from the shear center of the section, will induce a torsion in the section. The shear center of a singly-symmetric channel section (such as a typical AISC C or MC channel shape) is generally located at mid-depth of the section, but offset from the web in a direction opposite to the direction in which the flanges are pointed; in other words, the locations of the center of gravity and the shear center are not coincident in a channel shape.

Some of the other member types could potentially be used in the substructure of a routine I-girder bridge and the provisions of this Article would likely be applicable in such cases, but the provisions related to those particular member types are designated as beyond the scope of superstructure design. Otherwise, the provisions of this Article are not applicable.

6.12.1.2.3b Circular Tubes and Round HSS

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article are used to calculate the nominal flexural shear resistance, V_n, and the nominal torsional resistance, T_n, at the strength limit state of circular tubes, including round HSS (Hollow Structural Sections). V_n and T_n are used in the basic relationships provided in Article 6.12.1.2.3a to check the factored shear and torsional resistance of these members at the strength limit state (see the Discussion of Article 6.12.1.2.3a in this Guide).

The provisions of this Article are not applicable to the routine steel I-girder bridges covered by this Guide as circular tubes and round HSS are not used in these bridges, including as cross-frame members. These member types could potentially be used in the substructure of a routine I-girder bridge and the provisions of this Article would likely be applicable in such cases, but the provisions related to those particular member types are designated as beyond the scope and not applicable to superstructure design.

These members are not used in routine I-girder bridges, including as cross-frame or diaphragm members.

6.12.1.2.4 Special Provisions for HSS Members

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article are used to define the web depth, *D*, and inside flange width, b_{fi}, for the design of square and rectangular HSS (Hollow Structural Section) members, and the design wall thickness, *t*, for the design of square, rectangular, and round HSS members. This Article is referred to in Articles 6.9.4.2, 6.12.1.2.3, 6.12.2.2.2, and 6.12.2.2.3.

The provisions of this Article are not applicable to the routine steel I-girder bridges covered by this Guide as HSS members are not used in these bridges, including not being used as cross-frame members.

6.12.2 Nominal Flexural Resistance

6.12.2.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

This Article simply states that provisions for lateral-torsional buckling need not be considered when determining the nominal flexural resistance of the following miscellaneous member types covered in Article 6.12: composite members (i.e., concrete-encased shapes and circular concrete-filled steel tubes); noncomposite I- and H-shaped members bent about their weak axis (i.e., y-axis); and noncomposite circular tubes.

The provisions of this Article are not applicable to the routine steel I-girder bridges covered by this Guide as these miscellaneous member types are not used in these bridges. These member types could potentially be used in the substructure of a routine I-girder bridge and the provisions of this Article would likely be applicable in such cases, but the provisions related to those particular member types are designated as beyond the scope of superstructure design.

6.12.2.2 Noncomposite Members

6.12.2.2.1 I- and H-Shaped Members

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of this Article and subarticles quantify the nominal flexural resistance, M_n, of noncomposite I- and H-shaped members subject to flexure about their weak axis (i.e., their y-axis). I- and H-shaped members bent about their weak axis do not experience lateral-torsional buckling or web local buckling; the only limit states to be considered are yielding and flange local buckling. These provisions are also applicable to channels (see the Discussion of Article 6.12.2.2.5 in this Guide) and members consisting of two channel flanges connected by a web plate subject to flexure about their weak axis. The limit of $1.6F_yS_y$ in Eq. 6.12.2.2.1b-1 is intended to avoid substantial early yielding in channels subjected to weak axis bending, potentially leading to inelastic response under service conditions. The weak-axis plastic moment capacity of I-sections rarely exceeds this limit. For H-shaped members, M_p about the weak axis is equal to $1.5F_yS_y$. For cases where I- and H-shaped members are subject to flexure about their strong axis (the axis perpendicular to the web), this Article states that the provisions of Article 6.10 shall apply.

The routine steel I-girder bridges covered by this Guide may utilize I-shaped members or channels as a top chord in an end diaphragm or as a diaphragm for a shallow-depth rolled beam structure, but these members are not typically subjected to loading producing flexure about their weak axis (nor should they be). As such, the provisions of this Article do not apply to their design.

These provisions may be applicable to the design of I- or H-shaped piles that could potentially be used in the substructure for a routine steel I-girder bridge, but the design of piles is designated as beyond the scope of superstructure design. Therefore, the provisions of this Article are not applicable.

6.12.2.2.2 Rectangular Box-Section Members

6.12.2.2.2a General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article points to subsequent articles for the provisions related to the flexural design of homogeneous and hybrid doubly and singly symmetric single-cell rectangular noncomposite box-section members with or without longitudinal stiffeners bent about either principal axis in which the cross-section principal axes are parallel to the cross-section component plates. The provisions also apply for the flexural design of square and rectangular HSS (Hollow Structural Sections).

The routine steel I-girder bridges covered by this Guide are not comprised of noncomposite rectangular box-section flexural members, including HSS members; therefore, the provisions of this Article are not applicable.

6.12.2.2.2b Cross-Section Proportion Limits

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article specify cross-section proportion limits for webs and compression flanges with and without longitudinal stiffeners, tension flanges, the outside width of the section, and flange extensions on compression flanges for noncomposite rectangular box-section members subject to flexure.

The routine steel I-girder bridges covered by this Guide are not comprised of noncomposite rectangular box-section flexural members; therefore, the provisions of this Article are not applicable.

6.12.2.2.2c Classification of Sections with a Longitudinally Unstiffened Compression Flange

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article provide the necessary classification of noncomposite rectangular box-section members with a longitudinally unstiffened compression flange subject to flexure based on the web and compression-flange slenderness, and define several important cross-section based parameters for each classification employed in the calculation of the member flexural resistance in Article 6.12.2.2.2e (see the Discussion of Article 6.12.2.2.2e in this Guide).

The routine steel I-girder bridges covered by this Guide are not comprised of noncomposite rectangular box-section flexural members; therefore, the provisions of this Article are not applicable.

6.12.2.2.2d Classification of Sections with a Longitudinally Stiffened Compression Flange

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article provide the necessary classification of noncomposite rectangular box-section members with a longitudinally stiffened compression flange subject to flexure, define an effective area, A_{eff}, representing the longitudinally stiffened compression flange used to compute the yield moment of the effective section with respect to the compression flange and other effective section properties, and define several important cross-section based parameters for each classification employed in the calculation of the member flexural resistance in Article 6.12.2.2.2e (see the Discussion of Article 6.12.2.2.2e in this Guide).

The routine steel I-girder bridges covered by this Guide are not comprised of noncomposite rectangular box-section flexural members; therefore, the provisions of this Article are not applicable.

6.12.2.2.2e General Yielding, Compression Flange Local Buckling and Lateral Torsional Buckling

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article quantify the nominal flexural resistance, M_n, of noncomposite rectangular box-section members considering the combined effects of general yielding, compression-flange local buckling, and lateral-torsional buckling as a function of the unbraced length, the effective section properties, and the cross-section based parameters for each cross-section classification determined in Article 6.12.2.2.2c or 6.12.2.2.2d, as applicable (see the Discussion of Articles 6.12.2.2.2c and 6.12.2.2.2d in this Guide).

The routine steel I-girder bridges covered by this Guide are not comprised of noncomposite rectangular box-section flexural members; therefore, the provisions of this Article are not applicable.

6.12.2.2.2f Service and Fatigue Limit States and Constructibility

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article contains provisions that must be satisfied at the service and fatigue limit states and for constructibility for noncomposite rectangular box-section members subject to flexure. Shear in the webs is limited to the shear-yield or shear-bucking resistance, V_{cr}, under the factored load for constructibility specified in Article 3.4.2.1, and also under the unfactored permanent load plus the Fatigue I load combination to alleviate any significant flexing of the web due to shearing action. Post-buckling resistance is assumed at the strength limit state in computing the nominal flexural resistance of noncomposite box-section members with slender webs and/or noncompact or slender flanges or plate panels. Under the provisions of Article 6.9.4.5, these particular elements are investigated to check that plate local buckling does not occur under conditions producing maximum longitudinal compressive stress acting at one or both longitudinal edges of the element under consideration at the service limit state or during construction (see the Discussion of Article 6.9.4.5 in this Guide). The flange flexural stresses in the box-section member are also limited under the Service II load combination to help control permanent deformations of the member.

The routine steel I-girder bridges covered by this Guide are not comprised of noncomposite rectangular box-section flexural members; therefore, the provisions of this Article are not applicable.

6.12.2.2.2g Flange Effective Width or Area Accounting for Shear Lag Effects

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article provide simplified rules regarding the consideration of shear lag effects, where required, in noncomposite rectangular box-section members subject to flexure in lieu of a more refined analysis. Reductions to the effective compression-flange area and gross tension flange area are specified to account for shear lag effects in the computation of the flexural resistance of the member at the strength limit state and in the computation of the elastic flexural stresses at the service and fatigue limit states and for constructibility. The specified reductions are not intended to be applied within the bridge structural analysis.

The routine steel I-girder bridges covered by this Guide are not comprised of noncomposite rectangular box-section flexural members; therefore, the provisions of this Article are not applicable.

6.12.2.2.3 Circular Tubes and Round HSS

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article quantify the nominal flexural resistance, M_n, of noncomposite circular tubes, including round HSS (Hollow Structural Sections). The flexural resistance is taken as the smaller resistance based on yielding and local buckling, as applicable. A maximum D/t ratio is also specified for circular tubes used as flexural members, where D is the outside diameter of the tube and t is the thickness of the tube.

The routine steel I-girder bridges covered by this Guide do not utilize noncomposite circular tubes or round HSS; therefore, the provisions of this Article are not applicable.

6.12.2.2.4 Tees and Double Angles

6.12.2.2.4a General

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article quantify the nominal flexural resistance, M_n, of tees (WT) and double angles loaded in the plane of symmetry formed by their y-axis. The flexural resistance is taken as the smaller resistance based on yielding, lateral-torsional buckling, flange local buckling, and local buckling of tee stems and double-angle web legs determined in subsequent articles, as applicable (see the Discussion of Articles 6.12.2.2.4b through 6.12.2.2.4e in this Guide). Legs of double angles in continuous contact or with separators may together be assumed as double-angle web legs in applying these provisions. Flexure of these members about the y-axis is not a consideration for the routine steel I-girder bridges covered by this Guide.

The computation of M_n for tees and double angles is typically required for application in the appropriate interaction equations of Article 6.9.2.2.1 when these members are subject to eccentric axial compression (see the Discussion of Article 6.9.2.2.1 in this Guide).

For the routine steel I-girder bridges covered by this Guide, these provisions are applicable if a tee or double angle is used as a cross-frame member. It should be noted that using rolled steel tee (WT) and double-angle sections as cross-frame members for routine steel I-girder bridges is generally discouraged, while the use of single-angle sections is encouraged. The magnitude of the forces in cross-frame members in routine steel I-girder bridges covered by this Guide is generally small enough such that single-angle members are adequate. Cross-frame forces are typically only large enough to warrant the use of rolled steel tee (WT) or double-angle sections in curved and/or skewed steel I-girder bridges. Rolled steel tee (WT) sections are typically quite expensive to fabricate. Tee (WT) sections are cut from full wide-flange (W) shapes and generally require straightening after the cutting process, which adds significant fabrication effort and cost. Double-angle sections are often viewed as problematic from a maintenance perspective; the surfaces between the adjacent angle flanges are difficult or impossible to paint in the field, and/or can suffer from potentially severe pack rust.

6.12.2.2.4b Yielding

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article quantify the nominal flexural resistance, M_n, of tee (WT) and double-angle members based on yielding for cases where the tee stems or double-angle web legs are subject to tension or compression. The nominal flexural resistance, M_n, is defined as the plastic moment, M_p. However, upper limits are specified on the value of M_p that may be used in each case, i.e., depending on whether the tee stems or double-angle web legs are subject to tension or compression, to indirectly control situations where significant yielding of the stem or web legs may occur at service load levels.

For the routine steel I-girder bridges covered by this Guide, these provisions are applicable if a tee or double angle is used as a cross-frame member. It should be noted that using rolled steel tee (WT) and double-angle sections as cross-frame members for routine steel I-girder bridges is generally discouraged, while the use of single-angle sections is encouraged. The magnitude of the forces in cross-frame members in routine steel I-girder bridges covered by this Guide is generally small enough such that single-angle members are adequate. Cross-frame forces are typically only large enough to warrant the use of rolled steel tee (WT) or double-angle sections in curved and/or skewed steel I-girder bridges. Rolled steel tee (WT) sections are typically quite expensive to fabricate. Tee (WT) sections are cut from full wide-flange (W) shapes and generally require straightening after the cutting process, which adds significant fabrication effort and cost. Double-angle sections are often viewed as problematic from a maintenance perspective; the surfaces between the adjacent angle flanges are difficult or impossible to paint in the field, and/or can suffer from potentially severe pack rust.

6.12.2.2.4c LTB

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article quantify the nominal flexural resistance, M_n, of tee (WT) and double-angle members based on lateral-torsional buckling (LTB) for cases where the tee stems or double-angle web legs are subject to tension or compression. For all cases, the moment-gradient modifier, C_b, is not included in the equations for reasons discussed in the Commentary for this Article. The lateral-torsional buckling capacity is significantly greater for the case where the tee stem or double-angle web legs are in tension than for the case where the stem or web legs are in compression. Additionally, detailing of end connections for cases where the tee stem or double-angle web legs are in tension should be done in a manner to minimize the potential for fixed-end moments that induce compression in the stem or web legs.

For the routine steel I-girder bridges covered by this Guide, these provisions are applicable if a tee or double angle is used as a cross-frame member. It should be noted that using rolled steel tee (WT) and double-angle sections as cross-frame members for routine steel I-girder bridges is generally discouraged, while the use of single-angle sections is encouraged. The magnitude of the forces in cross-frame members in routine steel I-girder bridges covered by this Guide is generally small enough such that single-angle members are adequate. Cross-frame forces are typically only large enough to warrant the use of rolled steel tee (WT) or double-angle sections in curved and/or skewed steel I-girder bridges. Rolled steel tee (WT) sections are typically quite expensive to fabricate. Tee (WT) sections are cut from full wide-flange (W) shapes and generally require straightening after the cutting process, which adds significant fabrication effort and cost. Double-angle sections are often viewed as problematic from a maintenance perspective; the surfaces between the adjacent angle flanges are difficult or impossible to paint in the field, and/or can suffer from potentially severe pack rust.

6.12.2.2.4d Flange Local Buckling

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article quantify the nominal flexural resistance, M_n, of tee (WT) and double-angle members based on flange local buckling for cases where the tee flanges or double-angle flange legs are subject to compression. The elastic flange local buckling resistance equation for tee sections, i.e., Eq. 6.12.2.2.4d-2, need only be considered for fabricated tee sections as the flanges of rolled tee sections given in the AISC *Manual of Steel Construction* qualify as nonslender (see the Discussion of Article 6.9.4.2.1 in this Guide).

For the routine steel I-girder bridges covered by this Guide, these provisions are applicable if a tee or double angle is used as a cross-frame member. It should be noted that using rolled steel tee (WT) and double-angle sections as cross-frame members for routine steel I-girder bridges is generally discouraged, while the use of single-angle sections is encouraged. The magnitude of the forces in cross-frame members in routine steel I-girder bridges covered by this Guide is generally small enough such that single-angle members are adequate. Cross-frame forces are typically only large enough to warrant the use of rolled steel tee (WT) or double-angle sections in curved and/or skewed steel I-girder bridges. Rolled steel tee (WT) sections are typically quite expensive to fabricate. Tee (WT) sections are cut from full wide-flange (W) shapes and generally require straightening after the cutting process, which adds significant fabrication effort and cost. Double-angle sections are often viewed as problematic from a maintenance perspective; the surfaces between the adjacent angle flanges are difficult or impossible to paint in the field, and/or can suffer from potentially severe pack rust.

6.12.2.2.4e Local Buckling of Tee Stems and Double Angle Web Legs

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article quantify the nominal flexural resistance, M_n, of tee (WT) and double-angle members based on local buckling of the tee stems or double-angle web legs for cases where the stem or web legs are subject to compression.

For the routine steel I-girder bridges covered by this Guide, these provisions are applicable if a tee or double angle is used as a cross-frame member. It should be noted that using rolled steel tee (WT) and double-angle sections as cross-frame members for routine steel I-girder bridges is generally discouraged, while the use of single-angle sections is encouraged. The magnitude of the forces in cross-frame members in routine steel I-girder bridges covered by this Guide is generally small enough such that single-angle members are adequate. Cross-frame forces are typically only large enough to warrant the use of rolled steel tee (WT) or double-angle sections in curved and/or skewed steel I-girder bridges. Rolled steel tee (WT) sections are typically quite expensive to fabricate. Tee (WT) sections are cut from full wide-flange (W) shapes and generally require straightening after the cutting process, which adds significant fabrication effort and cost. Double-angle sections are often viewed as problematic from a maintenance perspective; the surfaces between the adjacent angle flanges are difficult or impossible to paint in the field, and/or can suffer from potentially severe pack rust.

6.12.2.2.5 Channels

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article quantify the nominal flexural resistance, M_n, of channels. The flexural resistance about the strong axis (i.e, the x-axis) is taken as the smaller resistance based on yielding or lateral-torsional buckling, as applicable. For channels in flexure about the weak axis (i.e., the y-axis), the provisions of Article 6.12.2.2.1 are to be applied with M_n limited to $1.6F_yS_y$, with λ_f taken as b_f/t_f where b_f is the full width of the flange and t_f is the flange thickness taken as the average flange thickness for rolled channels, to indirectly prevent substantial yielding of the member at service load levels. S_y is the elastic section modulus about the weak axis (i.e., the y-axis) (see the Discussion of Article 6.12.2.2.1 in this Guide). The equations specified for the lateral-torsional buckling resistance in this Article assume the channel is sufficiently braced at support locations to prevent twisting of the section at those points.

This Article also specifies flange and web slenderness limits for fabricated or bent-plate channels such that flange and web local buckling need not be checked. Rolled channels given in the AISC *Manual of Steel Construction* have compact flanges and webs for yield strengths not exceeding 65 ksi; therefore, these limits need not be checked for rolled channels but can be checked for completeness.

For the routine steel I-girder bridges covered by this Guide, these provisions are applicable when a channel is used as a top chord in an end diaphragm and it is designed as a flexural member to support the wheel loads coming onto the end of the deck, or when a channel is used as a diaphragm for a shallow-depth rolled beam structure.

6.12.2.2.6 Single Angles

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Single angles are not used as pure flexural members in the routine steel I-girder bridges covered by this Guide. The effect of the moments due to eccentricities on a single angle member may be ignored by designing the angle for eccentric axial compression using the effective slenderness ratio defined in Article 6.9.4.4 and by designing the angle for eccentric axial tension using the appropriate shear lag reduction factor, *U*, defined in Article 6.8.2.2, as applicable (see the Discussion of Articles 6.9.4.4 and 6.8.2.2 in this Guide). Therefore, the provisions of this Article are not applicable to the routine steel I-girder bridges covered by this Guide.

6.12.2.2.7 Rectangular Bars and Solid Rounds

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article quantify the nominal flexural resistance, M_n, of rectangular bars and solid rounds. The flexural resistance is taken as the smaller resistance based on yielding or lateral-torsional buckling, as applicable.

The routine steel I-girder bridges covered by this Guide do not utilize rectangular bars or solid rounds; therefore, the provisions of this Article are not applicable.

6.12.2.3 Composite Members

6.12.2.3.1 Concrete-Encased Shapes

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of this Article quantify the nominal flexural resistance, M_n, of concrete-encased steel shapes subject to flexure only or to flexure in combination with axial compression. The calculation of the nominal compressive resistance, P_n, of these members is covered in Article 6.9.5.1 (see the Discussion of Article 6.9.5.1 in this Guide). The members must satisfy the applicable limitations specified in Article 6.9.5.2 (see the Discussion of Article 6.9.5.2 in this Guide).

These members could potentially be used in substructures for the routine steel I-girder bridges covered by this Guide, although they are not commonly utilized. Therefore, these provisions are considered beyond the scope and not applicable to superstructure design.

6.12.2.3.2 Concrete-Filled Tubes

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of this Article quantify the nominal flexural resistance, M_n, of circular concrete-filled tubes or pipes where full composite action is not deemed necessary under combined axial compression and flexure. The calculation of the nominal compressive resistance, P_n, of these members is covered in Article 6.9.5.1 (see the Discussion of Article 6.9.5.1 in this Guide). For applications where full composite action is deemed necessary, the provisions of Article 6.12.2.3.3 should be employed instead (see the Discussion of Article 6.12.2.3.3 in this Guide). The members must satisfy the applicable limitations specified in Article 6.9.5.2 (see the Discussion of Article 6.9.5.2 in this Guide).

These members could potentially be used in substructures for the routine steel I-girder bridges covered by this Guide, although they are not commonly utilized. Therefore, these provisions are considered beyond the scope and not applicable to superstructure design.

6.12.2.3.3 Composite Concrete-Filled Steel Tubes (CFSTs)

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of this Article are used to develop the nominal flexural composite resistance, M_n, of composite circular concrete-filled steel tubes, or CFSTs, subject to flexure in combination with axial compression as a function of the nominal axial resistance, P_n. These larger-diameter members, which may be used for piers, columns, piles and drilled shafts, must satisfy the limitations specified in Article 6.9.6.2 (see the Discussion of Article 6.9.6.2 in this Guide). CFSTs should not be used as pure flexural members.

The nominal material-based nominal axial force/moment interaction diagram for these members is to be developed using either the Plastic Stress Distribution Method (PDSM) or the Strain Compatibility Method (SCM). The application of the PDSM to these members, which is a cross-section analysis using the constituent materials based on equilibrium at full plastification of the section, is described in detail in the Commentary for this Article. The resulting material-based interaction curve is modified as specified in Article 6.9.6.3.4 to include stability effects based on the buckling load determined in Article 6.9.6.3.2 to create a nominal stability-based interaction curve, which is then multiplied by the appropriate resistance factor specified in Article 6.5.4.2 to determine the final factored resistance of the CFST for combined axial compression and flexure for all load conditions (see the Discussion of Articles 6.9.6.3.2 and 6.9.6.3.4 in this Guide).

These members could potentially be used in substructures for the routine steel I-girder bridges covered by this Guide, although they are not commonly utilized. Therefore, these provisions are considered beyond the scope and not applicable to superstructure design.

6.12.3 Nominal Shear Resistance of Composite Members

6.12.3.1 Concrete-Encased Shapes

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of this Article quantify the nominal shear resistance, V_n, of concrete-encased steel shapes.

These members could potentially be used in substructures for the routine steel I-girder bridges covered by this Guide, although they are not commonly utilized. Therefore, these provisions are considered beyond the scope and not applicable to superstructure design.

6.12.3.2 Concrete-Filled Tubes

6.12.3.2.1 Rectangular Tubes

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of this Article quantify the nominal shear resistance, V_n, of rectangular concrete-filled tubes.

These members could potentially be used in substructures for the routine steel I-girder bridges covered by this Guide, although they are not commonly utilized. Therefore, these provisions are considered beyond the scope and not applicable to superstructure design.

6.12.3.2.2 Composite Concrete Filled Tubes

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of this Article quantify the nominal shear resistance, V_n, of circular concrete-filled steel tubes, including composite CFSTs both with and without longitudinal reinforcement.

These members could potentially be used in substructures for the routine steel I-girder bridges covered by this Guide, although they are not commonly utilized. Therefore, these provisions are considered beyond the scope and not applicable to superstructure design.

6.13 CONNECTIONS AND SPLICES

6.13.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

The provisions of this Article cover several general considerations related to connection design. Connections should be made symmetrical about the axis of the members, where practical. Members, including bracing, should be connected so that their gravity axes intersect at a point. Eccentric connections should be avoided, however, where this is not possible, the members and connections must be designed for the combined effects of the shear and moment due to the eccentricity.

Bolted connections, except for connections on lacing and handrails, are to contain not less than two bolts. In the case of connections that transfer total member end shear, the gross section is to be taken as the gross section of the connected elements. The calculation of individual bolt shear loads in cases where there is an in-plane eccentricity between the axis of loading and the centroid of the bolt group should consider the effects of both the direct shear force and the eccentric moment; methods for evaluating eccentrically-loaded bolted connections can be found in most steel design textbooks. When there is an out-of-plane eccentricity between the axis of loading and the plane of the bolted connection, the need to consider the effects of the eccentric moment should be considered on a case-by-case basis (see the Discussions of Articles 6.13.2.8 and 6.13.2.11 in this Guide).

The configuration of some welded end connections, such as the connection of a cross-frame member to a gusset plate, may result in an unbalanced weld condition, where the centroid of the weld group is offset from the centroid of the member, producing an eccentricity in the plane of the connection. For singly symmetric members (such as tee sections connected through the flange) or doubly symmetric members (rarely used for cross-frame members, but including I-shaped sections, HSS sections, etc.), a balanced condition is easily achieved by specifying equal-length longitudinal welds. For unsymmetrical members such as single-angle members, a balanced weld condition requires unequal length longitudinal welds. In other words, in the case of single angles, an eccentricity is created unless the weld group is balanced to align with the member centroid. To create a balanced condition for the welded connection of a single-angle member, the weld along the edge of the angle nearest the centroid of the angle would need to be longer than the weld along the opposite edge of the connected flange of the angle. Determining the weld geometry that achieves a balanced connection involves a relatively simple exercise in statics (two such methods are described below). However, detailing excessively oversized or odd-shaped gusset plates solely to allow for the unequal length longitudinal welds necessary to achieve a balanced weld configuration in this case is often impractical and/or uneconomical and is generally discouraged. Instead, consider evaluating the connection as an eccentrically loaded weld group, which causes additional shear loading of the welds. Descriptions of the procedures to determine balanced weld geometry or to evaluate the stresses in an unbalanced welded connection can be found in many steel design textbooks.

When faced with an unbalanced welded connection, where the centroid of the member is not aligned with the centroid of the weld group, the weld group should be designed for the additional moment generated by the eccentricity, which causes additional shear loading of the welds. There are two methods commonly recognized for analyzing eccentrically loaded welded connections: elastic (vector) analysis and the instantaneous center of rotation (strength) method. A brief explanation of each method is provided below. More detailed discussion of these methods can be found in many steel design textbooks.

Elastic analysis of an eccentrically loaded weld group assumes that each segment of weld resists a concentrically applied load with an equal force, and rotation is assumed to occur about the centroid of the weld configuration. The design moment (sometimes called the "torsional moment") is calculated by multiplying the axial force in the member by the eccentricity between the centroid of the member and the centroid of the welded connection. The load on each weld segment caused by the torsional moment is assumed to be proportional to the distance from the centroid. The direction of the force caused by torsion is assumed to be perpendicular to the radial distance from the centroid of the weld configuration. The components of the forces caused by direct load and torsion are combined vectorially to obtain a resultant force. This method is relatively simple to apply, but will provide conservative results, and can be excessively conservative for some conditions.

The instantaneous center of rotation method considers the combined effect of rotation and translation of one connection element with respect to the other. This combined effect is equivalent to a rotation about a point defined as the instantaneous center of rotation (IC). The location of the IC depends upon the geometry of the weld group as well as the direction and the point of application of the load. The individual resistance of each weld segment is assumed to be perpendicular to the radial distance from the IC. If the IC has been correctly located, the sum of the applied forces and the weld resistances about the IC will be zero. Locating the IC can be computationally cumbersome, and application of this method generally requires the use of tabulated values or an iterative solution. However, for connections where torsional effects are significant, this method provides more accurate and less conservative results than elastic analysis.

The effect of unbalanced welds on the strength and fatigue resistance of single-angle members can typically be neglected. The Commentary to Article J1.7 of AISC's Specifications for Structural Steel Buildings and Commentary states that tests have shown that the effect of slight eccentricities between the centroid of a welded connection and the centroid of the connected member have negligible effect on the static strength of the member. This same Commentary states, "However, the fatigue life of eccentrically loaded welded angles has been shown to be very short. Notches at the roots of fillet welds are harmful when alternating tensile stresses are normal to the axis of the weld, as could occur due to bending when axial cyclic loading is applied to angles with end welds not balanced about the neutral axis. Accordingly, balanced welds are required when such members are subjected to cyclic loading (see Figure C-J1.3)." However, this statement warrants some context. Research has shown that single-angle members connected to gusset plates with balanced welds generally exhibited slightly better fatigue performance than those with unbalanced, equal length welds, but all of the tested specimens, which consisted of angles with both balanced and unbalanced welds, "… failed within the E or E′ categories delineated by the AASHTO specification… using the stress range calculated on the gross section." The current AASHTO LRFD BDS treats the welded connection of angle and tee members to gusset plates as Category E′ details, and thus already captures the effect of an unbalanced welded connection on the fatigue resistance of the angle member.

Also note that historically it has been a common practice in a number of jurisdictions to specify only a total length of weld required for the connection of cross-frame members and allow the steel detailer to determine the specific weld geometry based on the orientation of the member and the gusset plate geometry. This practice is discouraged, as it may result in weld geometry which does not provide sufficient resistance for the anticipated design loads if an unbalanced condition, not anticipated by the designer, is detailed for the welds.

The effects of out-of-plane eccentricities on welded end connections should also be considered. In some cases, the effect of the out-of-plane eccentric moment on the weld stresses may be significant, in which case the total vector sum weld stress should be compared to the resistance of the weld. In other cases, similar to the Discussion of Article 6.13.2.1.1 for bolted connections, the out-of-plane eccentric moment may have only a negligible effect on the weld stresses in welded end connections if the length of the welds perpendicular to the eccentricity is large compared to the eccentric distance.

End connection angles used to connect stringers to floorbeams and/or floorbeams to girders should be connected with high-strength bolts. Where bolting is not practical, welded connections may be used, but they must be designed for the vertical loads and any bending moment resulting from restraint against end rotation. End connections for the stringers, floorbeams and girders should be made with two angles; doing so eliminates the need to account for the eccentricity of the stringer, floorbeam, or girder end reaction in the design of the perpendicular bolted connection. The bolted connection should however be designed for combined shear and tension (see the Discussion of Article 6.13.2.11 in this Guide). The thickness of end connection angles of stringers, floorbeams and girders is not to be less than 3/8 in. These provisions are applicable only to the design of girder-stringer-floorbeam systems and are not applicable to the routine steel I-girder bridges covered by this Guide.

The provisions in this Article related to the application of the 75 percent of the factored resistance or the average of the calculated factored axial force effect and the factored axial resistance apply solely to primary members subject only to axial tension or compression. In routine steel I-girder bridges, the only primary members are the I-shaped main spanning elements, which are considered “flexural members”. There are separate provisions (mentioned later in the Discussion of this Article) addressing connections and splices of primary flexural members. The other typical members in a routine steel I-girder bridge, such as the cross-frames or diaphragms, the bearing stiffeners, etc., are considered secondary members. As a result, the provisions related to the application of the 75 percent of the factored resistance, or the average of the calculated factored axial force effect and the factored axial resistance are not applicable to the design of routine steel I-girder bridges. The provisions of this Article related to connections and splices for primary members subjected to combined force effects are similarly not applicable to the routine steel I-girder bridges covered by this Guide.

Meanwhile, the provisions of this Article refer to the appropriate Articles for the design of bolted splices (Article 6.13.6.1.3) and welded splices (Article 6.13.6.2) for flexural members at the strength limit state, which *are* applicable to the routine steel I-girder bridges covered by this Guide (see the Discussion of Articles 6.13.6.1.3 and 6.13.6.2 in this Guide).

The provisions of this Article also specify that where cross-frames/diaphragms, lateral bracing, stringers or floorbeams for straight or horizontally curved members are included in a structural model used to determine force effects, or are designed for explicitly calculated force effects from the results of a separate investigation (e.g., an approximate wind load analysis), the end connections for those members are to be designed for the calculated factored member force effects. Otherwise, the end connections for these members are to be designed for 75 percent of the factored resistance corresponding to the force effect under consideration. Refined methods of analysis are not required, nor are they recommended, for the routine steel I-girder bridges covered by this Guide. However, it is recommended that reasonable cross-frame and diaphragm force effects be calculated using a "separate investigation." For the routine steel I-girder bridges which are the subject of this Guide (i.e., bridges which are straight, with little or no skew and limited span lengths), a reasonable "separate investigation" should include calculation of the following force effects:

- The member loads in the cross-frames or diaphragms resulting from the transfer of wind loads applied to the girders up to the deck
- The minimum strength requirements associated with the cross-frames or diaphragms functioning as stability bracing for the girders

As has been mentioned in the Discussion of Articles 6.7.4.1, 6.7.4.2, 6.8.1, and 6.8.2.1 in this Guide, it is required that in addition to the minimum design requirements specified in Article 6.7.4.1 (see the Discussion of Article 6.7.4.1 in this Guide), cross-frames or diaphragms for the routine steel I-girder bridges covered by this Guide also must be designed to satisfy the stability bracing strength and stiffness requirements specified in Article 6.7.4.2.2 (see the Discussion of Article 6.7.4.2.2 in this Guide).

The provisions in this Article related to the use of standard-size bolt holes in connections in horizontally curved bridges and timber stringers framing into steel floorbeams are not applicable to the routine steel I-girder bridges covered by this Guide.

6.13.2 Bolted Connections

6.13.2.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article contains some general considerations related to the design of bolted connections. Bolted steel parts must fit solidly together after the bolts are tightened. The bolted parts may be coated or uncoated. It must be specified in the contract documents that all joint surfaces, including surfaces adjacent to the bolt head and nut, be free of scale (except for tight mill scale), dirt or other foreign material. All material within the grip of the bolt must be steel.

High-strength bolts are to be installed to have a specified initial tension, which results in an initial precompression between the joined parts. At service load levels, the transfer of the loads between the joined parts may then occur entirely via friction with no bearing of the bolt shank against the side of the hole. Until the friction force is overcome, the shear resistance of the bolt and the bearing resistance of the bolt hole will not affect the ability to transfer the load across the shear plane between the joined parts.

The AASHTO LRFD BDS recognizes two types of high-strength bolted connections; slip-critical connections (see the Discussion of Article 6.13.2.1.1 in this Guide) and bearing-type connections (see the Discussion of Article 6.13.2.1.2 in this Guide). The resistance (or "strength") of high-strength bolted connections in transmitting shear across a shear plane between bolted steel parts is the same whether the connection is a slip-critical or bearing-type connection. The slip-critical connection has an additional requirement that sufficient frictional resistance be provided so that slip will not occur between the joined parts at service load levels.

For further information on high-strength bolts, including installation provisions and verification procedures, consult the AISC Design Guide 17 High Strength Bolts - A Primer for Engineers, the RCSC Specifications for Structural Joints Using High-Strength Bolts available from the Research Council on Structural Connections (RCSC), the AASHTO *LRFD Steel Bridge Fabrication Specifications*, the AASHTO *LRFD Bridge Construction Specifications*, and the applicable Owner-agency standards.

6.13.2.1.1 Slip-Critical Connections

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The provisions of this Article deal with the design of high-strength bolted slip-critical connections. When these connections are subject to shear, the load is transferred between the joined parts by friction up to a level of force that is dependent upon the clamping force and the coefficient of friction of the faying surfaces. The coefficient of friction depends on the faying surface condition, with mill scale, paint or other surface treatments determining the value of the friction coefficient.

Prior to joint slip, the bolts are not subject to shear nor are joined parts subject to bearing stress. Once the load exceeds the frictional resistance between the faying surfaces, slip occurs; that is, the friction bond is broken, and the two surfaces slip with respect to one another by a relatively large amount. Generally, a rupture failure does not occur when the shear loading in the connection exceeds the slip resistance; instead, the connection is able to continue resisting an even greater load through the shear resistance of the bolts and the bearing resistance against the connected material. Under this scenario, final failure of the connection is by shear failure of the bolts, yielding or tear-out of the connected material or by an unacceptable deformation around the holes. In other words, the ultimate resistance of the connection is not related to the slip load.

The slip and bearing resistances are computed separately for application at different load combinations, as described further below. Because the combined effect of frictional resistance with shear or bearing has not been systematically studied and is uncertain, any potential greater resistance due to combined effect is ignored. Because a high tensile preloading of the bolt (developed by properly installing the bolt to a prescribed torque or rotation) is required to develop a significant resisting friction force, only bolts with a high tensile yield strength (i.e., ASTM F3125

Grade A325 and A490 high-strength bolts and ASTM F3148 high-strength bolts – see the Discussion of Article 6.4.3.1.1 in this Guide) can be used in slip-critical connections.

According to the provisions of this Article, pretensioned high-strength bolted joints located where stress and strain due to joint slippage would be detrimental to the serviceability of the structure or may adversely affect the geometry of the structure are to be designated in the contract documents as "slip-critical" (the Engineer is referred to this Article for the complete list of joints that should be designated as slip-critical). Bolted field splices and connections utilizing oversize or slotted holes in routine steel I-girder bridges (which is not recommended) are to always be designed as slip-critical connections to control permanent deformations that could adversely affect the geometry of the structure.

Joints of diaphragm and cross-frame members in routine steel I-girder bridges with pretensioned high-strength bolts installed in standard holes should be designed only as bearing-type connections (see the Discussion of Article 6.13.2.1.2 in this Guide) as field experience has indicated that slip is not likely and that any slip that may occur in these connections is not anticipated to be detrimental to the geometry or serviceability of the structure.

Pretensioned high-strength bolted connections in routine steel I-girder bridges that are designed as slip-critical connections, i.e., diaphragm or cross-frame members that are provided with oversize or slotted holes in the connections (which is not recommended for routine steel I-girder bridges) and bolted field splices, are to be proportioned to prevent slip under the factored loads during the deck placement as specified in Article 6.10.3.1. Wind load for the active work zone condition during the deck placement as defined in the *AASHTO Guide Specifications for Wind Loads on Bridges During Construction* should also be considered in proportioning such connections in diaphragm and cross-frame members. To control permanent deformations under overloads caused by slip in joints that could adversely affect the serviceability of the structure, these slip-critical connections are also to be proportioned to prevent slip under Load Combination Service II, as specified in Table 3.4.1-1, and to provide bearing, shear, and tensile resistance at the applicable strength limit state load combinations assuming the connection has slipped and gone into bearing against the connected material. In addition, the resistance of the connected material must be checked at the strength limit state (See the Discussion of Articles 6.13.2.8, 6.13.2.7, 6.13.2.9, and 6.13.5 in this Guide for information on the calculation of the slip, shear, and bearing resistances of bolted connections and the resistance of the connected material, respectively). Lateral bracing members provided with oversize holes in one ply to aid in fit-up are exempted from the slip requirement under Load Combination Service II as the consequence of any slip in the connections of these members is not likely to be detrimental to the serviceability of the structure under this load condition.

For further information on high-strength bolts, including installation provisions and verification procedures, consult the AISC Design Guide 17 High Strength Bolts - A Primer for Engineers, the RCSC Specifications for Structural Joints Using High-Strength Bolts available from the Research Council on Structural Connections (RCSC), the AASHTO *LRFD Steel Bridge Fabrication* Specifications, the AASHTO *LRFD Bridge Construction Specifications* , and the applicable Owner-agency standards.

6.13.2.1.2 Bearing-Type Connections

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The provisions of this Article deal with the design of high-strength bolted bearing-type connections. In high-strength bolted bearing-type connections, the load is resisted by a combination of the shear resistance of the bolt, the bearing resistance of the connected material and an unknown amount of friction between the faying surfaces. The failure of a bearing-type connection will be by shear failure of the bolts, yielding or tear-out of the connected material or by an unacceptable deformation around the holes, with the final failure load independent of the clamping force provided by the bolts.

This Article specifies that bearing-type connections are only permitted for joints subject to axial compression or for joints of diaphragm, cross-frame, or lateral bracing members in routine steel I-girder bridges with high-strength bolts installed in standard holes (which is recommended – see the Discussion of Article 6.13.2.1.1 in this Guide). Connections utilizing ASTM A307 bolts are also to be designed as bearing-type connections. Such connections are to be designed to provide the required factored resistance in shear and bearing at the strength limit state. Faying surfaces of bearing-type connections need not satisfy the surface condition preparation specified in Article 6.13.2.8 for slip-critical connections (see the Discussion of Article 6.13.2.8 in this Guide); coatings are permitted in bearing-type connections.

Typically, of the situations outlined above, only joints in bracing members are found on the routine steel I-girder bridges covered by this Guide. Compression-only connections are unlikely, and A307 non-pretensioned bolts are rarely, if ever, used in structural connections in steel girder bridges. High-strength bolted bearing-type connections are to be fully pretensioned; the typical practice has been to use pretensioned high strength bolts (Grade ASTM A325 bolts or ASTM F3148 bolts – see the Discussion of Article 6.4.3.1.1 in this Guide) installed in standard holes in cross-frame and diaphragm connections on routine steel I-girder bridges, and such practice is recommended by this Guide.

For further information on high-strength bolts, including installation provisions and verification procedures, consult the AISC Design Guide 17 High Strength Bolts - A Primer for Engineers, the RCSC Specifications for Structural Joints Using High-Strength Bolts available from the Research Council on Structural Connections (RCSC), the AASHTO *LRFD Steel Bridge Fabrication* Specifications, the AASHTO *LRFD Bridge Construction Specifications*, , and the applicable Owner-agency standards.

6.13.2.2 Factored Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The provisions of this Article are used to determine the factored resistance of a bolted connection at the strength limit state (Eq. 6.13.2.2-2) and the factored resistance of an individual bolt under the Service II load combination (Eq. 6.13.2.2-1). The resistance factor is implicitly taken equal to

1.0 in the AASHTO LRFD BDS at the service limit state and therefore is not shown in the resistance equation for the Service II load combination. Although not stated, Eq. 6.13.2.2-1 should also be used to compute the factored resistance of an individual bolt in a bolted field splice under the factored loading used to investigate slip during the deck casting.

These provisions are generally applicable to the bolted connections in the routine steel I-girder bridges covered by this Guide. However, in most situations routine steel I-girder bridges are not subject to externally applied tensile forces, at least not greater than their specified pretension, and so the provisions related to bolts subject to axial tension or combined axial tension and shear (i.e., Eq. 6.13.2.2-3) are only conditionally applicable.

An example of a situation where bolts might be subject to axial tension (and in fact would likely be subject to combined axial tension and shear) would be the connection of a diaphragm to the web of a rolled steel beam using a partial-depth bolted angle connection. Such a detail is permitted in Article 6.6.1.3.1, which allows the connection of intermediate diaphragms on rolled beams in straight bridges with composite reinforced concrete decks whose supports are normal or are skewed less than 10 degrees from normal, and where those diaphragms are placed in contiguous lines parallel to the supports (see the Discussion of Article 6.6.1.3.1 in this Guide). In such a case, the bolted connection to the web would be subject to an out-of-plane moment inducing tension in some of the bolts connecting the angle to the web.

Alternately, in most cases, the bolted connections in routine steel I-girder bridges are subject primarily to shear, with only minor moments due to eccentricities between the connection and the centroid of the connected member. In most cases, the dimensions of the connection are large enough and the eccentricity is small enough that it is unlikely that the bolts will be subject to a net externally applied tensile force larger than their pretension. Typical cases where checking combined axial tension and shear in bolts is not necessary include bolted field splice connections and truss-type cross-frame connections.

In general, high-strength bolted connections designed according to the these provisions will have a higher reliability at the strength limit state than the connected parts because the resistance factors for the design of bolted connections were selected to provide a higher level of reliability than those chosen for member design. Also, the controlling strength limit state in the connected part, e.g., yielding or deflection, is typically reached well before the controlling strength limit state in the connection, e.g., the bolt shear resistance or the bearing resistance of the connected material.

6.13.2.3 Bolts, Nuts, and Washers

6.13.2.3.1 Bolts and Nuts

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The provisions of this Article simply refer to the provisions of Article 6.4.3 for the material specifications for high-strength bolts and nuts to be used in the bolted connections of the routine steel I-girder bridges covered by this Guide (see the Discussion of Article 6.4.3 in this Guide).

6.13.2.3.2 Washers

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The provisions of this Article simply refer to the provisions of Article 17.7.4.1 of the AASHTO *LRFD Steel Bridge Fabrication Specifications* and to the provisions of Article 6.4.3.1.3 for the material specifications and other requirements for hardened washers to be used in the bolted connections of the routine steel I-girder bridges covered by this Guide (see the Discussion of Article 6.4.3.1.3 in this Guide).

The provision in this Article related to direct tension indicators (DTIs) installed over an oversize or slotted hole in an outer ply applies only if DTIs are used in such connections. DTIs are washers which include mechanical features (typically small arch-shaped protrusions) which compress in response to the pretension developed in the bolt. When correctly calibrated, the amount of pretension can be determined by measuring the gap remaining between the washer and the connected element. The use of DTIs is subject to Owner-agency preferences; check Owner-agency policies and specifications before requiring or allowing their use.

6.13.2.4 Holes

6.13.2.4.1 Type

6.13.2.4.1a General

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article indicates that standard-size bolt holes (see the Discussion of Article 6.13.2.4.2 in this Guide) are to be used in high-strength bolted connections, unless otherwise specified. This provision is applicable to the bolted connections in the routine steel I-girder bridges covered by this Guide.

6.13.2.4.1b Oversize Holes

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article indicates that oversize bolt holes (see the Discussion of Article 6.13.2.4.2 in this Guide) may be used in any or all plies of slip-critical connections but are not to be used in bearing-type connections (see the Discussion of Articles 6.13.2.1.1 and 6.13.2.1.2 in this Guide).

For bolted field splices, the use of slip-critical connections is required, and the use of oversize holes is not permitted (see the Discussion of Article 6.13.6.1.3a in this Guide).

For cross-frame and diaphragm connections in the routine steel I-girder bridges covered by this Guide, the use of oversize holes is neither required nor prohibited by the AASHTO LRFD BDS. However, the typical practice has been to use pretensioned high strength bolts installed in standard holes in cross-frame and diaphragm connections on routine steel I-girder bridges, and such practice is recommended by this Guide. Such connections should be designed as bearing-type connections (see the Discussion of Articles 6.13.2.1.1 and 6.13.2.1.2 in this Guide).

Article 6.13.2.1.1 requires that, "Pretensioned high-strength bolted joints located where stress and strain due to joint slippage would be detrimental to the serviceability of the structure or may adversely affect the geometry of the structure shall be designated in the contract documents as slip-critical connections." That Article also requires that "joints in shear with bolts installed in oversize holes" are to be designated as slip-critical. Designation of a joint as slip-critical triggers fabrication requirements which specify the required conditions for the faying surfaces of the connection and the use of pretensioned high-strength bolts, and additional design requirements (see the Discussion of Article 6.13.2.1.1 in this Guide). Designers should consult Owner-agency policy regarding the use of standard or oversize holes for cross-frame or diaphragm connections in routine steel I-girder bridges. In the absence of such policy, the use of standard-size holes is recommended.

The use of oversize holes in other bolted connections in routine steel I-girder bridges should be evaluated on a case-by-case basis, comparing the value of facilitating easy fit-up of the structure versus the value of maintaining tighter control of the constructed geometry of the structure.

6.13.2.4.1c Short-Slotted Holes

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article indicates that short-slotted holes (see the Discussion of Article 6.13.2.4.2 in this Guide) may be used in any or all plies of either slip-critical or bearing-type connections (see the Discussion of Articles 6.13.2.1.1 and 6.13.2.1.2 in this Guide). In slip-critical connections, the slots may be used without regard to the direction of loading. However, in bearing-type connections, the length of the slot must be normal to the direction of the load.

Short-slotted holes are not typically used, nor are they recommended for use, on bolted connections in the routine steel I-girder bridges covered by this Guide. However, they may occasionally be used or necessary in special situations, such as phased construction or widening projects.

Vertical slotted holes for cross-frame connections have sometimes been used in straight skewed I-girder bridges in an attempt to minimize the twist of the girders and reduce the cross-frame forces. Such an approach is not recommended, however, as it becomes difficult to control the vertical deflections during the deck placement.

6.13.2.4.1d Long-Slotted Holes

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article indicates that long-slotted holes (see the Discussion of Article 6.13.2.4.2 in this Guide) may be used in only one ply of either slip-critical or bearing-type connections (see the Discussion of Articles 6.13.2.1.1 and 6.13.2.1.2 in this Guide). In slip-critical connections, the slots may be used without regard to the direction of loading. However, in bearing-type connections, the length of the slot must be normal to the direction of the load.

Long-slotted holes are not typically used, nor are they recommended for use, on bolted connections in the routine steel I-girder bridges covered by this Guide. However, they may occasionally be used or necessary in special situations, such as phased construction or widening projects.

Vertical slotted holes for cross-frame connections have sometimes been used in straight skewed I-girder bridges in an attempt to minimize the twist of the girders and reduce the cross-frame forces. Such an approach is not recommended, however, as it becomes difficult to control the vertical deflections during the deck placement.

6.13.2.4.2 Size

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article provides the maximum permitted size of standard, oversize, short-slotted and long-slotted holes in bolted connections (Table 6.13.2.4.2-1). This Article is applicable to the routine steel I-girder bridges covered by this Guide.

In Table 6.13.2.4.2-1, *d* is the diameter of the bolt. Standard-size holes for bolts with diameters less than 1 in. are 1/16-in. larger than the *d*. Oversize holes for bolts with diameters less than 1 in. are 3/16-in. larger than *d*. Standard-size holes for bolts with diameters greater than or equal to 1 in. are 1/8-in. larger than *d*, which avoids the need to field ream holes to fit large-diameter hot-forged bolts, which often have a longitudinal forging seam that interferes with holes 1/16 in. larger than the bolt diameter.

Article 6.8.3 specifies that for the calculation of the net area in design calculations, the width of standard bolt holes is to be taken as the nominal diameter of the hole (see the Discussion of Article 6.8.3 in this Guide). The width of oversize and slotted holes is to be taken as the nominal diameter or width of the hole, as applicable, given in Table 6.13.2.4.2-1.

6.13.2.5 Size of Bolts

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article gives some specific requirements regarding the size of bolts in bolted connections. These requirements are applicable to the routine steel I-girder bridges covered by this Guide. Bolts are not to be less than 0.625 in. in diameter. Bolts 0.625 in. in diameter are not to be used in primary members, except for 2.5-in. legs of angles and in flanges of sections whose dimensions require 0.625-in. bolts to satisfy other detailing provisions given in the specifications. Structural shapes that do not permit the use of 0.625-in. bolts are to be limited to use in handrails.

6.13.2.6 Spacing of Bolts

6.13.2.6.1 Minimum Spacing and Clear Distance

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article provides minimum spacing and clear distance requirements for bolted connections, which are applicable to the routine steel I-girder bridges covered by this Guide.

The minimum spacing between centers of bolts in standard holes in any direction is not to be less than 3.0d, where d is the diameter of the bolt. The minimum clear distance, L_c, between the edges of adjacent bolt holes in the direction of the force and transverse to the direction of the force is not to be less than 2.0d when oversize or slotted holes are used.

6.13.2.6.2 Maximum Spacing for Sealing Bolts

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article provides maximum spacing requirements for sealing bolts in bolted connections, which are applicable to the routine steel I-girder bridges covered by this Guide. These requirements are intended to help seal against the penetration of moisture in joints and apply to the lines of bolts in the connection adjacent to a free edge of an outside plate or shape.

Maximum spacing requirements are specified for a single line of bolts (Eq. 6.13.2.6.2-1), and for the staggered spacing in two lines considered together where there is a second line of bolts uniformly staggered with the line adjacent to the free edge at a gage distance less than $1.5 + 4.0t$ (Eq. 6.13.2.6.2-2). The staggered spacing need not be less than one-half the requirement for a single line. The thickness, t, in these requirements is the thickness of the thinner outside plate or shape. The absolute maximum spacing in both instances is 7.0 in.

In uncoated weathering steel structures, it is critical that the bolt spacing be such that the connection joint is tight and excess moisture cannot enter between the plies of material. If sufficient moisture enters the joint, the resulting corrosion may cause prying, or pack-out, of the joint or bolt failure. The maximum bolt spacing requirements for sealing bolts have been shown to provide proper tightness and stiffness of uncoated weathering steel bolted joints to avoid joint prying and corrosion pack-out.

6.13.2.6.3 Maximum Pitch for Stitch Bolts

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article presents maximum pitch requirements for stitch bolts, which fasten together built-up compression or tension members at intervals along their length, such that the separate components will act as a unit to resist buckling of compression members. Requirements are given for a single line of bolts (12t for compression members and 24t for tension members) and for two adjacent lines of staggered bolts (Eq. 6.13.2.6.3-1 for compression members and twice the value from that equation for tension members). The gage between adjacent lines of bolts is not to exceed 24*t* for compression and tension members. The thickness, *t*, in these requirements is the thickness of the thinner outside plate or shape. The pitch is not to exceed the maximum pitch specified for sealing bolts.

These provisions are only applicable to mechanically fastened built-up members subject to axial compression or tension. These types of members are not used in routine steel I-girder bridges; therefore, the provisions of this Article are not applicable.

6.13.2.6.4 Maximum Pitch for Stitch Bolts at the End of Compression Members

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article provides more stringent maximum pitch requirements for stitch bolts at the ends of mechanically fastened built-up members subject to axial compression. The pitch, *p*, of the stitch bolts must not exceed 4.0*d* for a length equal to 1.5 times the maximum width of the member, where *d* is the diameter of the bolt. Beyond this length, *p* may be increased gradually over a length equal to 1.5 times the maximum width of the member until the maximum pitch given by either 12.0*t* or Eq. 6.13.2.6.3-1, as applicable, is reached (see the Discussion of Article 6.13.2.6.3 in this Guide). The thickness, *t*, in these requirements is the thickness of the thinner outside plate or shape.

These provisions are only applicable to mechanically fastened built-up members subject to axial compression. These types of members are not used in routine steel I-girder bridges; therefore, the provisions of this Article are not applicable.

6.13.2.6.5 End Distance

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article specifies the minimum and maximum end distance requirements for all types of holes in bolted connections and are applicable to the bolted connections in the routine steel I-girder bridges covered by this Guide.

The end distance of bolts is defined in the AASHTO LRFD BDS as the distance along the line of force between the center of a hole and the end of the component. This Article specifies that the end distance for all types of holes is not to less than the appropriate minimum edge distance specified in Table 6.13.2.6.6-1 (see the Discussion of Article 6.13.2.6.6 in this Guide). When oversize or slotted holes are used, the minimum clear end distance, which is defined as the distance between the edge of the bolt hole and the end of the member, must not be less than the bolt diameter.

The maximum end distance is to be taken the same as the maximum edge distance, or the lesser of eight times the thickness of the thinnest outside plate and 5.0 in.

6.13.2.6.6 Edge Distances

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article specifies the minimum and maximum edge distance requirements for all types of holes in bolted connections and are applicable to the bolted connections in the routine steel I-girder bridges covered by this Guide.

The edge distance of bolts is defined in the AASHTO LRFD BDS as the distance perpendicular to the line of force between the center of a hole and the edge of the component. The minimum edge distance is a function of the diameter of the bolt. This Article specifies that the minimum edge distance is to be taken as specified in Table 6.13.2.6.6-1.

The maximum edge distance is not to be more than the lesser of eight times the thickness of the thinnest outside plate and 5.0 in.

The minimum edge distances are based on standard fabrication practices and workmanship tolerances. Providing edge (and end) distances larger than the specified minimum edge distances, but not larger than the specified maximum edge distances, is encouraged as it allows for the occurrence of minor fabrication errors (due to unavoidable workmanship tolerances) without violating the specified minimum distances. Also, satisfying the provisions of Article 6.13.2.9 related to the bearing resistance of bolt holes (see the Discussion of Article 6.13.2.9 in this Guide) provides sufficient end distances such that bearing and tear-out limits are not exceeded for bolts adjacent to all types of edges.

The maximum edge distance limits are intended to help seal the faying surfaces. In uncoated weathering steel structures, it is critical that the connection joint is tight and excess moisture cannot enter between the plies of material. If sufficient moisture enters the joint, the resulting corrosion may cause prying, or pack-out, of the joint or bolt failure. The maximum edge distance limits have been shown to provide proper tightness and stiffness of uncoated weathering steel bolted joints to avoid joint prying and corrosion pack-out.

6.13.2.7 Shear Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The provisions of this Article are used to calculate the nominal shear resistance, R_n, of bolts at the strength limit state and are applicable to the connections in the routine steel I-girder bridges covered by this Guide. Note that the nominal shear resistance of anchor rods is covered separately in Article 6.13.2.12 (see the Discussion of Article 6.13.2.12 in this Guide). In slip-critical connections (see the Discussion of Article 6.13.2.1.1 in this Guide), it is assumed that the bolts in the connection have slipped and gone into bearing at the strength limit state such that the shear resistance of the bolts and bearing resistance at the bolt holes must be checked.

The nominal shear resistance depends on whether the threads are excluded from (Eq. 6.13.2.7-1) or included in (Eq. 6.13.2.7-2) the shear plane. Eqs. 6.13.2.7-1 and 6.13.2.7-2 apply for determining the nominal shear resistance of a high-strength bolt (ASTM F3125 bolt) or an ASTM A307 bolt at the strength limit state in joints whose length between extreme fasteners measured parallel to the line of action of the force is less than or equal to 38.0 in. For a bolt in lap-splice tension and compression connections greater than 38.0 in. in length, the nominal shear resistance is to be taken as 0.83 times the value given by Eq. 6.13.2.7-1 or 6.13.2.7-2, as applicable. The nominal shear resistance is based on the observation that the shear strength of a single high-strength bolt is about 0.625 times the tensile strength F_{ub} of the bolt (see Table 6.4.3.1.1-1 and the Discussion of Article 6.4.3.1.1 in this Guide for the determination of F_{ub}).

The shear resistance is not affected by the pretension in the bolts provided the connected material is in contact at the faying surfaces. In lap-splice tension and compression connections with more than two bolts in the line of force, the average bolt strength decreases as the joint length increases due to the nonuniform bolt shear force distribution caused by deformation of the connected material. For joints up to 38.0 in. in length, a single reduction factor of 0.90 is implicitly applied to the 0.625 multiplier described above rather than providing a function that reflects the decrease in average bolt strength with joint length (0.90 times 0.625 equals the 0.56 multiplier given in Eq. 6.13.2.7-1). This was felt not to adversely affect the economy of very short joints. For bolts in joints longer than 38.0 in., the nominal shear resistance must be reduced by an overall reduction factor of 0.75. Therefore, the nominal shear resistance of bolts in joints longer than 38.0 in. must be further reduced by an additional factor of 0.83 or 0.75/0.90. For bolted flange splices, note that the 38.0 in. length is to be measured between the extreme bolts on only one side of the connection. The potential application of the additional joint length reduction factor of 0.83 applies only to lap splice tension and compression connections and not to web shear-type connections; e.g., web splices. The reduction factor of 0.90 is retained for web shear-type connections since the distribution of the bolt forces is not necessarily uniform. However, a further reduction in the shear resistance based on the joint length is not necessary. When bolts are positioned so that they cross two planes of contact (i.e., N_s = 2), this is referred to as ′double shear′. Double shear is a symmetrical loading situation with regard to the shear planes and direction of shear transfer. When there is a single plane of contact involved in the load transfer (i.e., N_s = 1), this is referred to as ′single shear′, which is an unsymmetrical loading situation.

The average ratio of the nominal shear resistance for bolts with threads included in the shear plane to the nominal shear resistance for bolts with threads excluded from the shear plane is 0.83 with a standard deviation of 0.03. Therefore, a reduction factor of 0.80 is conservatively used to account for the nominal shear resistance when threads are included in the shear plane but calculated with the area corresponding to the nominal bolt diameter (0.56 * 0.80 equals the 0.45 multiplier given in Eq. 6.13.2.7-2).

In determining whether threads should be included or excluded from the shear plane, consideration should be given to the location of the shear plane relative to the transition length of the bolt. The transition length is the tapered area between the portion of the bolt that is fully threaded and the portion of the bolt where the shank is completely unthreaded. Prior to 2020, shear planes located in the transition length of high-strength bolts were traditionally treated as if the threads were excluded from the shear plane; however, the June 11, 2020 edition of the Research Council on Structural Connections (RCSC) Specification for Structural Joints Using High-Strength Bolts instituted a more conservative provision where shear planes located in the transition length of high-strength bolts should be considered as shear planes with the threads included. Consequently, the Commentary to Article 6.13.2.7 in the 10th Edition of the AASHTO LRFD BDS was updated to provide guidance on how to determine the limits of the transition length, considering normally accepted construction variations permitted in the manufacturing of bolts and various combinations of bolt, washers, and direct tension indicators (DTIs). The Commentary also discusses cases where shorter length bolts will be fully threaded. In general, experience has shown that in connections of thicker elements (such as the connections in the flange splices of bolted field splice details), threads will typically be excluded from the shear plane, while in connections of much thinner elements (such as the connections in the web splices of bolted field splice details), threads may be included in the shear plane. Clear trends may not exist for other cases (such as cross-frame gusset plate connections). Regardless of the trends, designers are encouraged to evaluate each connection and design accordingly.

In evaluating the limits of the transition length of the bolts in a given connection detail, designers are encouraged to exercise reasonably conservative judgment when not certain which direction the bolts may be oriented in a connection or when the specific use or location of washers and DTIs may be left to the discretion of the detailer. Designers are further encouraged to document their connection design assumptions on the plans but are discouraged from arbitrarily prohibiting the detailing of connections with threads in the shear plane when it is not reasonable to assume that the detailer and fabricator can achieve such a condition in a practical manner. For example, placing a note on the plans stating, "Threads shall be excluded from the shear plane," when an examination of the connection shows that short, fully threaded bolts are likely to be used, will not achieve the intent of having a connection with threads excluded from the shear plane. Instead, the detailer will likely submit an RFI requesting permission to detail the connection with threads in the shear plane to avoid other undesirable solutions such as providing multiple unnecessary washers to force the use of a longer bolt; dealing with such an RFI after award of the contract will be challenging for the designer as it may require a redesign of the connection.

Since the threaded length of an ASTM A307 bolt is not as predictable as that of a high-strength bolt, the nominal shear resistance of an A307 bolt must always be based on Eq. 6.13.2.7-2. Also, A307 bolts with a long grip (i.e., the total thickness of the plies of a joint through which the bolt passes exclusive of any washers or load-indicating devices) tend to bend, reducing their shear resistance. Therefore, this Article requires that when the grip length of an A307 bolt exceeds 5.0 bolt diameters, the nominal shear resistance must be lowered 1.0 percent for each 1/16 in. of grip in excess of 5.0 bolt diameters.

Bolted cross-frame gusset plate connections are often subject to eccentric shear; that is, when one of the resultant member forces is applied on a line of action that does not pass through the center of gravity of the bolt group. The resultant action may be represented as a net moment (torque) equal to the resultant force times its eccentricity from the center of gravity, and a concentric force acting on the connection. Since both the moment and concentric force cause shears on the bolt group, this particular situation is referred to as eccentric shear.

For further discussion and an example design calculation of eccentric shear, consult Section 6.6.4.2.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design

(LRFD) for Highway Bridge Superstructures; this topic is also addressed in most steel design textbooks.

For an example design calculation of the shear resistance of a high-strength bolt, consult Section 6.6.4.2.5.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.13.2.8 Slip Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The provisions of this Article are used to calculate the nominal slip resistance, R_n, of a bolt in a high-strength bolted slip-critical connection under Load Combination Service II, and in some instances, during the deck casting (see the Discussion of Article 6.13.2.1.1 in this Guide). These provisions are applicable to the slip-critical connections in the routine steel I-girder bridges covered by this Guide.

The bolt pretension and surface condition of the faying surface (i.e., coefficient of friction) have the greatest effect on the slip resistance of high-strength bolted connections. The minimum required bolt pretension during bolt installation, P_t, to be used in Eq. 6.13.2.8-1 is given for ASTM F3125 (with a specified minimum tensile strength of 120 or 150 ksi) and F3148 (with a specified minimum tensile strength of 144 ksi) high-strength bolts (see the Discussion of Article 6.4.3.1.1 in this Guide) in Table 6.13.2.8-1.

The surface condition factor, K_s, (i.e., coefficient of friction) to be used in Eq. 6.13.2.8-1 is provided in Table 6.13.2.8-3 and is a function of the class of the surface. Four different classes of surfaces (Classes A through D) are defined based on the mean value of slip coefficients from many tests of clean mill scale, uncoated and coated blast-cleaned steel surfaces, unsealed pure zinc or 85/15 zinc-aluminum thermal-sprayed (i.e., metalized) surfaces with coating thicknesses not exceeding 16 mils, hot-dip galvanized surfaces, and mixed faying surfaces utilizing an unsealed pure zinc thermal-sprayed coating mating with a hot-dip galvanized surface. The surface condition is typically defined, either directly or indirectly, in the Owner-agencies standard specifications.

It has been found that if tightly adherent mill scale is on the faying surface of a bolted connection on uncoated weathering steel, the connection slips into bearing at a lower shear stress than on a carbon steel with mill scale. However, if the faying surface is blast-cleaned to the Association for Materials Protection and Performance (AMPP) standard SP-6 (SSPC-SP-6) or better, slip-critical connections on uncoated weathering steel can be designed using a Class B surface condition. Otherwise, a Class A surface condition, which is appropriate for clean mill-scale surfaces, must be used. The slip resistance of bolted joints is not affected by the weathering of uncoated steel surfaces prior to erection, but any loose rust on the connection or faying surfaces must be removed. Pre-construction primers may be used for the cleaned bolted surfaces. Hot-dip galvanized faying surfaces need not be treated after galvanizing. Consult the Commentary for this Article for information on the effects of paint overspray and the required testing to qualify a particular coating to be used under these specifications.

Since faying surfaces (that are not galvanized) are typically blast-cleaned as a minimum, a Class A surface condition should only be used to compute the slip resistance when Class A coatings are applied or when unpainted mill scale is left on the faying surface. Most commercially available primers will qualify as Class B or Class D coatings.

Since all locations must develop the slip resistance before a total joint slip can occur at that plane, the assumption is made that the slip resistance at each bolt is equal and additive with the slip resistance at the other bolts in the connection. It is also assumed that the full slip resistances must be mobilized at each slip plane before full joint slip can occur, although the forces at each slip plane do not necessarily develop simultaneously. Eq. 6.13.2.8-1 is formulated for the case of a single slip plane. Therefore, the total slip resistance of a joint with multiple slip planes can be taken equal to the resistance of a single slip plane multiplied by the number of slip planes, N_s.

Hole size factors, K_h, in Eq. 6.13.2.8-1 less than 1.0 are provided for bolts in oversize or slotted holes (Table 6.13.2.8-2) because of the greater possibility of significant deformation occurring in joints with oversize or slotted holes. For long-slotted holes, even though the slip load is the same for bolts loaded transverse or parallel to the axis of the slot, the hole size factor for loading parallel to the axis has been reduced, based upon judgment, because of the greater consequences of slip in this case.

In a slip-critical connection subject to the combined effects of a net overall axial tension and shear, the tensile force reduces the contact pressure between the connected plates thereby reducing the slip resistance to the shear forces. The reduction in slip resistance is approximately proportional to the ratio of the applied tensile force to the bolt installation tension. The reduction factor is given by Eq. 6.13.2.11-3 (see the Discussion of Article 6.13.2.11 in this Guide). The local reduction in contact pressure due to an overturning moment causing a local tension in one part of a connection does not reduce the slip resistance since there is a corresponding increase in contact pressure in other parts of the connection.

In the 10th Edition of the AASHTO LRFD BDS, an additional factor, K_c, was introduced to account for the effect of creep in connections with Class C galvanized faying surfaced or duplex-coated faying surfaces utilizing a coating over a galvanized subsurface. The factor appears in Eq. 6.13.2.8-1 and reflects the potential loss of pretension over time due to creep in the compressible soft pure zinc surface layer; a reduction in bolt pretension would result in a corresponding reduction in slip resistance.

For an example design calculation of the slip resistance of a high-strength bolt, consult Section 6.6.4.2.4.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.13.2.9 Bearing Resistance at Bolt Holes

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The provisions of this Article are used to calculate the nominal bearing resistance at bolt holes, R_n, in a bolted connection at the strength limit state and are applicable to the connections in the routine steel I-girder bridges covered by this Guide. In slip-critical connections (see the Discussion of Article 6.13.2.1.1 in this Guide), it is assumed that the bolts in the connection have slipped and gone into bearing at the strength limit state such that the shear resistance of the bolts and bearing resistance at the bolt holes must be checked.

After a major slip has occurred in a slip-critical connection, one or more bolts are in bearing against the side of the hole. A bearing failure relates generally to either deformation of the bolt or deformation around a bolt hole. Tests have consistently shown that the bearing stress on the bolt itself is not critical. The contact pressure between the bolt and connected material can be expressed as the bearing stress on the connected material. For simplicity, the bearing stress is assumed to be a uniform stress distribution equal to the load transmitted by the bolt divided by the bearing area taken as the bolt diameter times the thickness of the connected material. The actual failure mode depends on the clear end distance or clear distance between the bolts, the bolt diameter, and the thickness of the connected material. Either the bolt will split out through the end of the plate because of insufficient end distance, or else excessive deformations are developed in the connected material adjacent to the bolt hole because of insufficient clear distance between the bolts.

The nominal bearing resistance of an interior hole is based on the clear distance, L_c, between the edge of the hole and the edge of the adjacent hole in the direction of the bearing force. The nominal bearing resistance of an end hole is based on the clear distance, L_c, between the edge of the hole and the end of the member. Eq. 6.13.2.9-1 or 6.13.2.9-2 is typically applicable for the routine steel I-girder bridges covered by this Guide as these equations apply to standard-size holes. These two equations can be combined and written more simply for design as $R_n = 1.2L_c tF_u \leq 2.4dtF_u$. Eqs. 6.13.2.9-3 and 6.13.2.9-4 apply only for the case of long-slotted holes perpendicular to the applied bearing force and can be combined similarly.

The design bearing resistance is expressed in terms of a single bolt, although it is truly for the connected material adjacent to the bolt. Since bearing failure is generally related to either deformation of the bolt or deformation around a bolt hole, rather than a fracture-type failure, the connection will continue to carry its failure load if subject a force greater than the bearing resistance. Therefore, in calculating the nominal bearing resistance for the connected part, the total bearing resistance may be taken as the sum of the bearing resistances of the individual bolts (holes) parallel to the line of the applied force. Also, if the nominal bearing resistance of a bolt hole exceeds the nominal shear resistance of the bolt determined as specified in Article 6.13.2.7 (see the Discussion of Article 6.13.2.7 in this Guide), the nominal bearing resistance of the bolt hole is to be limited to the nominal shear resistance of the bolt.

For an example design calculation of the bearing resistance of a connected part in a high-strength bolted connection, consult Section 6.6.4.2.5.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.13.2.10 Tensile Resistance

6.13.2.10.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article contains general provisions related to the tensile resistance of high-strength bolts. The Article specifies that high-strength bolts subject to axial tension must be pretensioned to the level given in Table 6.13.2.8-1 (see the Discussion of Article 6.13.2.8 in this Guide) regardless of whether the design is for a slip-critical or a bearing-type connection (see the Discussion of Articles 6.13.2.1.1 and 6.13.2.1.2 in this Guide).

Axial tension occurring without simultaneous shear occurs in bolts for tension members such as hangers or other members whose line of action is perpendicular to the member to which it is fastened. The applied tensile force must be taken as the force due to externally applied loads plus any tension resulting from prying action produced by deformation of the connected parts as specified in Article 6.13.2.10.4 (see the Discussion of Article 6.13.2.10.4 in this Guide).

Bolted connections in the routine steel I-girder bridges covered by this Guide generally are not subject to axial tension occurring without simultaneous shear, and so the provisions of this Article generally are not applicable. Of course, unique, specialty details may involve tension-only connections, but these types of details would be the exception rather than the rule in routine steel I-girder bridges.

6.13.2.10.2 Nominal Tensile Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article specifies the nominal tensile resistance of a bolt, T_n, independent of any initial tightening force. The tensile resistance of a bolt is the product of the tensile strength of the bolt and the tensile stress area through the threaded portion of the bolt. The tensile stress area is approximately 76 percent of the nominal cross-sectional area of the bolt for the usual sizes of a structural bolt. Hence, the nominal tensile resistance per unit area, based on the nominal area of the bolt, is taken as 76 percent of the tensile strength of the bolt.

The specified nominal tensile resistance is approximately equal to the initial tightening force specified in 6.13.2.8-1. Thus, when a tensile force is applied to a high-strength bolt that has been properly pretensioned, the increase in the bolt tension is generally much smaller than the applied load. There will be little increase in bolt force above the pretension load at service load levels. After the parts separate, the bolt will act as a tension member with the applied force equaling the bolt tension. As a result, bolts in connections subject to axial tension are required to be fully pretensioned.

Bolted connections in the routine steel I-girder bridges covered by this Guide generally are not subject to axial tension occurring without simultaneous shear, and so the provisions of this Article generally are not applicable. Of course, unique, specialty details may involve tension-only connections, but these types of details would be the exception rather than the rule in routine steel I-girder bridges.

6.13.2.10.3 Fatigue Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article deal with the fatigue resistance of high-strength bolts subject to axial tension. Properly pretensioned high-strength bolts subject to fatigue in axial tension must satisfy Eq. 6.6.1.2.2-1 (see the Discussion of Article 6.6.1.2.2 in this Guide). The stress range (Δf) in the equation is to be taken as the stress range in the bolt due to the passage of the 72-kip fatigue design load (plus the 15 percent dynamic load allowance) specified in Article 3.6.1.4 (see the Discussion of Article 3.6.1.4 in this Guide), plus any prying force resulting from the cyclic application of the fatigue load (see the Discussion of Article 6.13.2.10.4 in this Guide); the initial tension in the bolts is not to be included. The stress range is to be computed using the nominal diameter of the bolt.

In calculating the nominal fatigue resistance $(\Delta F)_n$ from Eq. 6.6.1.2.5-1 or 6.6.1.2.5-2, as applicable, the detail category constant A and the constant-amplitude fatigue threshold $(\Delta F)_{TH}$ for ASTM F3125 Grade A325 and A490 bolts (with a specified minimum tensile strength of 120 ksi and 150 ksi, respectively) and ASTM F3148 bolts (with a specified minimum tensile strength of 144 ksi) in axial tension are to be taken directly from Table 6.6.1.2.3-1 (Condition 9.3) when the bolts are fully pretensioned. Otherwise, Condition 9.4 in Table 6.6.1.2.3-1 applies (see the Discussion of Articles 6.6.1.2.3 and 6.6.1.2.5 in this Guide).

This Article limits the calculated prying force to 30 percent of the externally applied load when bolts are subject to tensile fatigue loading. This limit is based on limited investigations of prying effects under fatigue loading. Since low carbon ASTM A307 bolts are of lower strength and are not pretensioned, this Article prohibits their use in connections subjected to fatigue loading.

Bolted connections in the routine steel I-girder bridges covered by this Guide generally are not subject to axial tension occurring without simultaneous shear, and so the provisions of this Article generally are not applicable. Of course, unique, specialty details may involve tension-only connections, but these types of details would be the exception rather than the rule in routine steel I-girder bridges.

6.13.2.10.4 Prying Action

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article contains a conservative simplified expression to calculate the tensile force due to prying action in bolted connections subjected to axial tension. Prying action is defined as a lever action that exists in connections subjected to axial tension in which the line of application of the applied load is eccentric to the axis of the bolt, causing deformation of the fitting and an amplification of the axial force in the bolt. Part 9 of the AISC *Manual of Steel Construction* provides a more comprehensive treatment of prying action.

Bolted connections in the routine steel I-girder bridges covered by this Guide generally are not subject to pure axial tension and prying action, but in some cases bolted connections may be subject to combined axial tension and shear, and depending on the particular circumstances of the detail, consideration of prying action may be warranted. As a result, the provisions of this Article are considered conditionally applicable.

An example of a situation where bolts might be subject to axial tension (and in fact would likely be subject to combined axial tension and shear) would be the connection of a diaphragm to the web of a rolled steel beam using a partial-depth bolted angle connection. Such a detail is permitted in Article 6.6.1.3.1, which allows the connection of intermediate diaphragms on rolled beams in straight bridges with composite reinforced concrete decks whose supports are normal or are skewed less than 10 degrees from normal, and where those diaphragms are placed in contiguous lines parallel to the supports (see the Discussion of Article 6.6.1.3.1 in this Guide). In such a case, the bolted connection to the web would be subject to an out-of-plane moment inducing tension in some of the bolts connecting the angle to the web. Furthermore, depending on the specific detailing used, the bolts may also be subject to prying action.

6.13.2.11 Combined Tension and Shear

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article deals with the resistance of high-strength bolts in connections subjected to combined axial tension and shear; e.g., less than full-depth diaphragm end-angle bolted connections to webs and end-angle bolted connections of stringers to floorbeams and/or floorbeams to girders.

Eqs. 6.13.2.11-1 and 6.13.2.11-2 are simplifications of elliptical interaction curves that provide the nominal tensile resistance of bolts subjected to combined axial tension and shear. No reduction in the nominal tensile resistance is required when the applied factored shear force on the bolt is less than or equal to 33 percent of the nominal shear resistance of the bolt (see the Discussion of Article 6.13.2.7 in this Guide).

In a slip-critical connection subject to the combined effects of a net overall axial tension and shear, the tensile force reduces the contact pressure between the connected plates thereby reducing the slip resistance to the shear forces (see the Discussion of Article 6.13.2.8 in this Guide). The reduction in slip resistance given by Eq. 6.13.2.11-3 is approximately proportional to the ratio of the applied tensile force to the bolt installation tension. The local reduction in contact pressure due to an overturning moment causing a local tension in one part of a connection does not reduce the slip resistance since there is a corresponding increase in contact pressure in other parts of the connection.

In routine steel I-girder bridges, there are only a few limited cases where bolted connections may be subject to combined axial tension and shear. As a result, the provisions of this Article are considered conditionally applicable.

An example of a situation where bolts might be subject to axial tension (and in fact would likely be subject to combined axial tension and shear) would be the connection of a diaphragm to the web of a rolled steel beam using a partial-depth bolted angle connection. Such a detail is permitted in Article 6.6.1.3.1, which allows the connection of intermediate diaphragms on rolled beams in straight bridges with composite reinforced concrete decks whose supports are normal or are skewed less than 10 degrees from normal, and where those diaphragms are placed in contiguous lines parallel to the supports (see the Discussion of Article 6.6.1.3.1 in this Guide). In such a case, the bolted connection to the web would be subject to an out-of-plane moment inducing tension in some of the bolts connecting the angle to the web. Furthermore, depending on the specific detailing used, the bolts may also be subject to prying action (see the Discussion of Article 6.13.2.10.4 in this Guide).

A converse example of a situation where consideration of axial tension in a bolted connection can generally be safely neglected might be the bolted connection of a gusset plate to a cross-frame connection plate (stiffener). Although some of the bolts in this connection may be subjected to combined tension and shear due to the out-of-plane moment arising from the eccentricity in the connection, the eccentricities and cross-frame forces in a routine steel I-girder bridge are small enough that this effect is unlikely to be significant and may be ignored in these bridges.

In the 10th Edition of the AASHTO LRFD BDS, an additional factor, K_c, was introduced to account for the effect of creep in connections with Class C galvanized faying surfaced or duplex-coated faying surfaces utilizing a coating over a galvanized subsurface. The factor appears in Eq. 6.13.2.11-3 and reflects the potential loss of pretension over time due to creep in the compressible soft pure zinc surface layer; a reduction in bolt pretension would result in a corresponding reduction in slip resistance.

6.13.2.12 Shear Resistance of Anchor Rods

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article are used to calculate the nominal shear resistance, R_n, of an ASTM F1554 anchor rod at the strength limit state. These provisions are applicable if anchor rods are used to connect bearing sole plates or masonry plates to the substructure.

Since the thread length of anchor rods is not limited by the specifications, threads are assumed included in the shear plane. The implicit joint length reduction factor of 0.90 is also not applicable to anchor rods (see the Discussion of Article 6.13.2.7 in this Guide). Therefore, the coefficient in the expression for R_n given by Eq. 6.13.2.12-1 is equal to the shear strength-to-tensile strength ratio multiplier of 0.625 times the reduction factor of 0.80 for threads included in the shear plane, or 0.50 (see the Discussion of Article 6.13.2.7 in this Guide).

The provisions do not cover the global design of the anchorages to the concrete, which is beyond the scope of this Guide.

6.13.3 Welded Connections

6.13.3.1 General

Determination of applicability, *Simple Span Bridges*: Applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Applicable.

Discussion:

The provisions for welded connections in the AASHTO LRFD BDS are certainly applicable to the design of routine steel I-girder bridges where welded steel plate girders are used as the main spanning elements, since these girders necessarily use welds to connect the flanges to the webs, as well as other details such as stiffeners and so on. The provisions may also be applicable to the design of routine steel I-girder bridges where rolled steel beams are used as the main spanning elements, if welded stiffeners or other welded connection details are involved with the diaphragms or other features.

Welding is the process of joining two pieces of material, usually metals, by heating the pieces to a suitable temperature such that the materials are soft enough to coalesce or fuse into one material. The pieces are held in position for welding and may or may not be pressed together depending on the process that is used. Arc welding, in which electrical energy in the form of an electric arc is introduced to generate the heat necessary for welding, is the most commonly used process in the steel-bridge construction industry. The heat of the electric arc as the current passes through the system simultaneously melts a consumable electrode (deposited as filler material) and the parts of the material being joined, with the joint resulting from the cooling and solidification of the fused material.

To protect the molten region from impurities, the zone to be welded is typically blanketed in an atmosphere supplied by a flux, which may be a fusible coating on the welding rod, a fusible powder spread over the line of the weld or a gas sprayed over the weld. To produce a weld of the desired quality, the properties of the electrode must be carefully controlled. Proper control of the current and voltage along with a skilled welder are also required to produce a quality weld.

The provisions of this Article refer to the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* (BWC) for requirements related to base metal, weld metal, and welding design details. The provisions also refer to AWS A2.4 *Standard Symbols for Welding, Brazing, and Nondestructive Examination* for information on welding symbols.

These provisions also require the use of matching weld metal in groove and fillet welds, with the exception that undermatching weld metal may be used in fillet welds if the welding procedure and weld metal are selected to facilitate the production of sound welds.

Weld metal strength may be classified as matching or undermatching. Matching weld (filler) metal has a specified minimum tensile strength that is the same as or higher than the lower-strength base metal. For example, matching weld metal for ASTM A709 Grade 50 steel would be E70 filler material, where the specified minimum weld/base metal properties tensile strength are 70/65 ksi. Although the weld metal has slightly higher properties than the base metal in this case, this is considered to be a matching combination. Matching strengths for various weld and base metal combinations are specified in the BWC.

According to these provisions, the use of undermatching weld metal is strongly encouraged for fillet welds connecting steels with specified minimum yield strength greater than 50 ksi. Lower strength weld metal will generally be more ductile than higher strength weld metal. Since the residual stresses in a welded connection are assumed to be on the order of the yield point of the weaker material in the connection, using lower strength weld metal will lower the level of residual stresses in the base metal at the connection reducing the cracking tendencies. Therefore, undermatching welds will be much less sensitive to delayed hydrogen cracking and are more likely to consistently produce sound welds. In such cases, the Engineer should indicate where undermatching welds are acceptable on the contract drawings. The use of undermatching weld metal is not applicable to the routine steel I-girder bridges covered by this Guide, as steels in these bridges are assumed to be either Grade 36 or Grade 50 steels.

These provisions are applicable to the simple span and multi-span continuous rolled beam bridges covered by this Guide if welded connections are used for the bracing connections or for bearing stiffeners (if required – see the Discussion of Article D6.5 in this Guide).

The FHWA Bridge Welding Reference Manual is an excellent resource for additional information on weld metal, welding processes, welding design details, and the appropriate designations of welding symbols and consumables. Extensive guidance on the design of welded connections for steel girder bridges can be found in Section 6.6.4.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.13.3.2 Factored Resistance

6.13.3.2.1 General

Determination of applicability, *Simple Span Bridges*: Applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Conditionally applicable.Conditionally applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Applicable.

Discussion:

The provisions for welded connections in the AASHTO LRFD BDS are certainly applicable to the design of routine steel I-girder bridges where welded steel plate girders are used as the main spanning elements, since these girders necessarily use welds to connect the flanges to the webs, as well as other details such as stiffeners and so on. The provisions may also be applicable to the design of routine steel I-girder bridges where rolled steel beams are used as the main spanning elements, if welded stiffeners or other welded connection details are involved with the diaphragms or other features.

The provisions of this Article point the Engineer to the appropriate Articles for determining the factored resistance, R_r, of the welded connections (see the Discussions of Articles 6.13.3.2.2 through 6.13.3.2.4 of this Guide) and the connected material at the strength limit state (see the Discussion of Article 6.13.5 in this Guide. The provisions also point to the Article for determining the effective area of the weld (see the Discussion of Article 6.13.3.3 in this Guide).

The factored resistance of a welded connection is based on either the factored resistance of the base metal, or the product of the deposited weld metal strength and the effective area of the weld that resists the load. The weld metal strength is the capacity of the weld metal itself, typically given in units of ksi. The effective area of the weld that resists the load is the product of the effective length and the effective throat of the weld (see the Discussion of Article 6.13.3.3 in this Guide). The factored resistance of the connected material is governed by the thickness of the connected parts.

The classification strength of the weld metal, F_{exx}, is taken as the specified minimum tensile strength of the weld metal in ksi, which is reflected in the classification designation of the electrode. For example, the ′70′ in E70XX (SMAW), ER70S (solid-wire GMAW), and E70C (metal-cored GMAW); and the ′7′ in E71XX (FCAW) and F7XX (SAW) in the classification designation of the electrodes for the indicated welding processes indicate a specified minimum tensile strength of 70.0 ksi.

These provisions are applicable to the simple span and multi-span continuous rolled beam bridges covered by this Guide if welded connections are used for the bracing connections or for bearing stiffeners (if required – see the Discussion of Article D6.5 in this Guide).

The FHWA Bridge Welding Reference Manual is an excellent resource for additional information on weld metal, welding processes, welded design details, and the appropriate designations of welding symbols and consumables. Extensive guidance on the design of welded connections for steel girder bridges can be found in Section 6.6.4.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS. Consult the AASHTO-NSBA Steel Bridge Collaboration Guidelines G1.4-2006 Guidelines for Design Details and G12.1-2020 Guidelines to Design for Constructability and Fabrication for recommendations, commentary, and sample welded design details that allow for more economical fabrication and erection.

6.13.3.2.2 Complete Joint Penetration Groove-Welded Connections

6.13.3.2.2a Tension and Compression

Determination of applicability, *Simple Span Bridges*: Applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Applicable.

Discussion:

The provisions for welded connections in the AASHTO LRFD BDS are certainly applicable to the design of routine steel I-girder bridges where welded steel plate girders are used as the main spanning elements, since these girders necessarily use welds to connect the flanges to the webs, as well as other details such as stiffeners and so on. The provisions may also be applicable to the design of routine steel I-girder bridges where rolled steel beams are used as the main spanning elements, if welded stiffeners or other welded connection details are involved with the diaphragms or other features.

Complete joint penetration (CJP) groove welds are most often used to connect structural members aligned in the same plane (i.e., butt joints such as flange and web shop splices). They can also be used in tee and corner joints, although CJP groove welds are not recommended for use in these joints because of the relatively high cost and resulting welding deformations in tee joints, and the fact that backing bars must typically be left in place in corner joints. CJP groove welds have the same resistance as the pieces joined and are intended to transmit the full load of the members joined.

CJP groove welds may be single- or double-sided welds. Double-sided welds, which require access to both sides of the joint, may require less weld metal and result in less distortion and are of particular importance when joining thick members. Groove welds are classified according to their shape. Most groove welds require a specific edge preparation and are named accordingly. The selection of the proper groove weld is dependent on the cost of the edge preparations, the welding process used, and the cost of making the weld. The decision as to which groove type to use is usually left to the Fabricator/Detailer, who will select the type of groove that will generate the required quality at a reasonable cost. The Engineer need only indicate on the contract drawings that a CJP groove weld is required at a particular joint.

In groove welds, the maximum forces are usually tension or compression. According to the provisions of this Article, the factored resistance, R_r, of CJP groove-welded connections subjected to tension or compression normal to the effective area or parallel to the axis of the weld at the strength limit state is to be taken as the factored resistance of the base metal. Tests have shown that groove welds of the same thickness as the connected parts are adequate to develop the factored resistance of the connected parts.

These provisions are not applicable to the simple span and multi-span continuous rolled beam bridges covered by this Guide as CJP groove-welded connections (e.g., flange and web welded butt (shop) splices) are not typically used in these bridges.

Consult the FHWA Bridge Welding Reference Manual is an excellent resource for additional information on the types of welded connections and their design. Extensive guidance on the design of welded connections for steel girder bridges can be found in Section 6.6.4.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS. Consult the AASHTO-NSBA Steel Bridge Collaboration Guidelines G1.4-2006 Guidelines for Design Details and G12.1-2020 Guidelines to Design for Constructability and Fabrication for recommendations, commentary, and sample welded design details that allow for more economical fabrication and erection.

6.13.3.2.2b Shear

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

Complete joint penetration (CJP) groove welds are most often used to connect structural members aligned in the same plane (i.e., butt joints such as flange and web shop splices). They can also be used in tee and corner joints, although CJP groove welds are not recommended for use in these joints because of the relatively high cost and the resulting welding deformations in tee joints, and the fact that backing bars must typically be left in place in corner joints. CJP groove welds have the same resistance as the pieces joined and are intended to transmit the full load of the members joined.

CJP groove welds may be single- or double-sided welds. Double-sided welds, which require access to both sides of the joint, may require less weld metal and result in less distortion and are of particular importance when joining thick members. Groove welds are classified according to their shape. Most groove welds require a specific edge preparation and are named accordingly. The selection of the proper groove weld is dependent on the cost of the edge preparations, the welding process used, and the cost of making the weld. The decision as to which groove type to use is usually left to the Fabricator/Detailer, who will select the type of groove that will generate the required quality at a reasonable cost. The Engineer need only indicate on the contract drawings that a CJP groove weld is required at a particular joint.

The provisions of this Article deal with the computation of the factored resistance, R_r, of CJP groove-welded connections subjected to shear on the effective area of the weld. In groove welds, the maximum forces are usually tension or compression. CJP groove-welded connections are generally not subjected to shear in the routine steel I-girder bridges covered by this Guide. However, some Owner-agency policies or standard details use CJP welds to connect bearing stiffeners, that are also being used as cross-frame or diaphragm connection plates, to the girder flange; in those cases, the provisions of this Article may be applicable. Generally, a detailed evaluation of this type of CJP weld application is not necessary though, since the other connection plates on the bridge should be connected to the flanges using fillet welds and those connections would typically control. Note that the use of CJP welds to connect bearing stiffeners to flanges is generally not required and is not recommended. A much more practical and economical detail is to provide a finish-to-bear condition alone (if the bearing stiffener does not additionally serve as a cross-frame or diaphragm connection plate) or a combination of finish-to-bear condition with fillet wells (if the bearing stiffener does additionally serve as a cross-frame or diaphragm connetion plate).

The FHWA Bridge Welding Reference Manual is an excellent resource for additional information on the types of welded connections and their design. Extensive guidance on the design of welded connections for steel girder bridges can be found in Section 6.6.4.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS. Consult the AASHTO-NSBA Steel Bridge Collaboration Guidelines G1.4-2006 Guidelines for Design Details and G12.1-2020 Guidelines to Design for Constructability and Fabrication for recommendations, commentary, and sample welded design details that allow for more economical fabrication and erection.

6.13.3.2.3 Partial Joint Penetration Groove-Welded Connections

6.13.3.2.3a Tension or Compression

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Partial joint penetration (PJP) groove welds do not extend completely through the thickness of the pieces being joined and are subject to special design requirements. PJP groove welds are sometimes used when stresses are low and there is no need to develop the complete strength of the base material. PJP groove welds can be used to connect structural members aligned in the same plane when the joints are subject to compression or shear only, provided adequate throats can be developed. They can also be used in tee and corner joints, particularly as larger throats are required, and are sometimes used in combination with fillet welds in these joints.

PJP groove welds may be single- or double-sided welds. Double-sided welds, which require access to both sides of the joint, may require less weld metal and result in less distortion and are of particular importance when joining thick members. Groove welds are classified according to their shape. Most groove welds require a specific edge preparation and are named accordingly. The selection of the proper groove weld is dependent on the cost of the edge preparations, the welding process used, and the cost of making the weld. When designing PJP welds, engineers are encouraged to select from the series of standard joints for PJP welds in Figure 4.5 of the BWC; such joints can be used on the Weld Process Specification (WPS) without the need for further testing to demonstrate the suitability of the joint itself. See Section 4.4, Standard Joints, in the FHWA Bridge Welding Reference Manual for further discussion of standard joints.

In groove welds, the maximum forces are usually tension or compression. The provisions of this Article deal with the computation of the factored resistance, R_r, of PJP groove-welded connections subjected to tension or compression parallel to the axis of the weld, and connections subjected to tension or compression normal to the effective area of the weld, at the strength limit state. Eq. 6.6.1.2.5-4 should also be considered in the fatigue design of PJP groove-welded connections subject to tension normal to the effective area of the weld (see the Discussion of Article 6.6.1.2.5 in this Guide).

PJP groove-welded connections are not used, nor are they recommended for use, in the routine steel I-girder bridges covered by this Guide; therefore, these provisions are not applicable. In many cases, it is more feasible and economical to use fillet welds instead of PJP groove welds.

The FHWA Bridge Welding Reference Manual is an excellent resource for additional information on the types of welded connections and their design. Extensive guidance on the design of welded connections for steel girder bridges can be found in Section 6.6.4.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS. Consult the AASHTO-NSBA Steel Bridge Collaboration Guidelines G1.4-2006 Guidelines for Design Details and G12.1-2020 Guidelines to Design for Constructability and Fabrication for recommendations, commentary, and sample welded design details that allow for more economical fabrication and erection.

6.13.3.2.3b Shear

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Partial joint penetration (PJP) groove welds do not extend completely through the thickness of the pieces being joined and are subject to special design requirements. PJP groove welds are sometimes used when stresses are low and there is no need to develop the complete strength of the base material. PJP groove welds can be used to connect structural members aligned in the same plane when the joints are subject to compression or shear only, provided adequate throats can be developed. They can also be used in tee and corner joints, particularly as larger throats are required, and are sometimes used in combination with fillet welds in these joints.

PJP groove welds may be single- or double-sided welds. Double-sided welds, which require access to both sides of the joint, may require less weld metal and result in less distortion and are of particular importance when joining thick members. Groove welds are classified according to their shape. Most groove welds require a specific edge preparation and are named accordingly. The selection of the proper groove weld is dependent on the cost of the edge preparations, the welding process used, and the cost of making the weld. When designing PJP welds, engineers are encouraged to select from the series of standard joints for PJP welds in Figure 2.5 of the BWC; such joints can be used on the Weld Process Specification (WPS) without the need for further testing to demonstrate the suitability of the joint itself. See Section 4.4, Standard Joints, in the FHWA Bridge Welding Reference Manual for further discussion of standard joints.

The provisions of this Article deal with the computation of the factored resistance, R_r, of PJP groove-welded connections subjected to shear parallel to the axis of the weld. PJP groove-welded connections are not used, nor are they recommended for use, in the routine steel I-girder bridges covered by this Guide; therefore, these provisions are not applicable. In many cases, it is more feasible and economical to use fillet welds instead of PJP groove welds.

The FHWA Bridge Welding Reference Manual is an excellent resource for additional information on the types of welded connections and their design. Extensive guidance on the design of welded connections for steel girder bridges can be found in Section 6.6.4.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS. Consult the AASHTO-NSBA Steel Bridge Collaboration Guidelines G1.4-2006 Guidelines for Design Details and G12.1-2020 Guidelines to Design for Constructability and Fabrication for recommendations, commentary, and sample welded design details that allow for more economical fabrication and erection.

6.13.3.2.4 Fillet-Welded Connections

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

Fillet welds are the most widely used welds due to their ease of fabrication and overall economy and are certainly the most widely used welds for bracing member, connection plate, stiffener, and flange-to-web connections; therefore, the provisions of this Article are applicable to the routine steel I-girder bridges covered by this Guide.

Fillet welds generally require less precision during fit-up and the edges of the joined pieces seldom need special preparation such as beveling or squaring. Fillet welds have a triangular cross-section and do not fully fuse the cross-sectional area of the parts they join, although full-strength connections can be developed with fillet welds. Lap joints utilize fillet welds and are the primary joint type for bracing member connections (e.g., bracing member-to-gusset plate joints). They are also used for flange-to-web welds in built-up sections and for connection plate and stiffener welds to webs and flanges. Fillet welds can also be used in tee and corner joints subject to shear.

The factored resistance of fillet welds is based on the assumption that failure of such welds is by shear on the effective area whether the shear transfer is parallel or perpendicular to the axis of the line of the weld. In fact, the resistance is greater for shear transfer perpendicular to the weld axis; however, for simplicity the situations are treated the same. Therefore, the factored resistance of fillet welds may be controlled by the shear resistance of the weld metal or by the shear rupture resistance of the connected material. Shear yielding is not critical in welds because the material strain hardens without large overall deformations occurring.

According to the provisions of this Article, the factored resistance, R_r, of fillet welds which are made with matched or undermatched weld metal and which have typical weld profiles at the strength limit state is to be taken as the smaller of the factored shear rupture resistance of the connected material adjacent to the weld leg determined as specified in Article 6.13.5.3 (see the Discussion of Article 6.13.5.3 in this Guide), and the product of the effective area specified in Article 6.13.3.3 (see the Discussion of Article 6.13.3.3 in this Guide) and the factored shear resistance of the weld metal given by Eq. 6.13.3.2.4-1, which depends on the classification strength, F_{exx}, of the weld metal (see the Discussion of Article 6.13.3.2.1 in this Guide). The factored shear rupture resistance of the base metal adjacent to the weld leg will seldom control since the effective area of the base metal at the weld leg is typically about 30 percent greater than the weld throat.

If fillet welds are subjected to eccentric loads that produce a combination of shear and bending stresses, they should be proportioned on the basis of a direct vector addition of the shear forces on the weld (consult Section 6.6.4.3.7.2.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures). Most of the welded connection details commonly used in routine steel I-girder bridges feature direct load paths which do not include significant eccentricity. An example of an unusual situation where a welded connection in a routine steel I-girder bridge might be subject to eccentric loading is the case of a cross-frame gusset plate welded to a cross-frame connection plate (stiffener). Although this type of connection detail is uncommon, at least one large Owner-agency prefers its use.

Also, see the Discussion of Article 6.13.1 in this Guide for a discussion of evaluating welded end connections for unbalanced weld conditions. Fillet-welded end connections of cross-frame members in routine steel I-girder bridges may exhibit unbalanced weld conditions; if so, the effects of the unbalanced geometry should be considered in the design of the connection.

The FHWA Bridge Welding Reference Manual is an excellent resource for additional information on the types of welded connections and their design. Extensive guidance on the design of welded connections for steel girder bridges can be found in Section 6.6.4.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. Consult the AASHTO-NSBA Steel Bridge Collaboration Guidelines G1.4-2006 Guidelines for Design Details and G12.1-2020 Guidelines to Design for Constructability and Fabrication for recommendations, commentary, and sample welded design details that allow for more economical fabrication and erection.

For specific design examples of connections utilizing fillet welds, consult Section 6.6.4.3.7.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.13.3.3 Effective Area

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The factored resistance of welds is based on the effective area of the weld, defined in this Article as the effective length of the weld times the effective throat. The effective throat is defined nominally as the shortest distance from the joint root to the face of the weld neglecting any weld reinforcement, or the minimum width of the expected failure plane.

The effective length of a groove weld is the width of the part joined perpendicular to the direction of stress. By definition, the effective throat of a CJP groove weld is equal to the thickness of the thinner part joined, with no increase allowed for any weld reinforcement. To achieve fusion throughout the thickness of the part being joined, backing is usually required if the CJP groove weld is made from one side, and back gouging is usually required from the second side if the CJP groove weld is made from both sides. Otherwise, qualification testing is required to show that the full throat can be developed.

The effective throat (effective groove weld size) of PJP groove welds is defined in Clause 4.3.1.3 and Annex A of the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* (BWC). The effective throat of PJP groove welds depends on the probable depth of fusion that will be achieved; that is, the depth of groove preparation and depth of penetration that can be achieved by the selected welding process and welding position. Minimum effective throat thickness requirements for PJP groove welds are also given in the BWC. PJP groove-welded connections are not used, nor are they recommended for use, in the routine steel I-girder bridges covered by this Guide.

The effective length of a fillet weld is to be taken as the overall length of the full-size fillet. The effective throat dimension of a fillet weld for a typical fillet weld with equal legs of nominal size, a, is taken equal to $0.707a$, or nominally the shortest distance from the joint root to the weld face, neglecting any reinforcement.

The FHWA Bridge Welding Reference Manual is an excellent resource for additional information on the types of welded connections and their design. Extensive guidance on the design of welded connections for steel girder bridges can be found in Section 6.6.4.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. Consult the AASHTO-NSBA Steel Bridge Collaboration Guidelines G1.4-2006 Guidelines for Design Details and G12.1-2020 Guidelines to Design for Constructability and Fabrication for recommendations, commentary, and sample design details that allow for more economical fabrication and erection.

For specific design examples of connections utilizing fillet welds, consult Section 6.6.4.3.7.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.13.3.4 Size of Fillet Welds

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The provisions of this Article state that the size of a fillet weld that may be assumed in the design of a connection is to be such that the forces due to the factored loadings do not exceed the factored resistance, R_r, of the connection specified in Article 6.13.3.2 (see the Discussion of Article 6.13.3.2 in this Guide). The size of a fillet weld is given as the leg size of the fillet.

These provisions also specify the maximum and minimum size requirements for fillet welds. Maximum thickness (size) requirements for fillet welds along edges of connected parts depend on the thickness of the parts being connected, unless the weld is specifically designated on the contract documents to be built out to obtain full throat thickness. The requirements prevent melting of the base metal where the fillet would meet the corner of the plate if the fillet were made the full plate thickness.

The minimum thickness (size) of a fillet weld is not to be less than that required to transmit the required forces, nor the minimum thickness specified in Table 6.13.3.4-1. The minimum weld size need not exceed the thickness of the thinner part joined. Note that the specified minimum weld sizes assume that the required preheats and interpass temperatures are provided (consult the FHWA Bridge Welding Reference Manual for additional information on preheats and interpass temperatures). Smaller welds than the minimum size welds may be approved by the Design Engineer if they are shown to be adequate for the applied stress and if the appropriate additional preheat is applied.

Minimum thickness requirements for fillet welds are based on preventing too rapid a rate of cooling to prevent a loss of ductility (i.e., the formation of a brittle microstructure) or a lack of fusion. The thicker the plate joined, the faster the heat is removed from the welding area. As a minimum, a weld of sufficient size is needed to prevent the thicker plate from removing heat at a faster rate than it is being supplied to cause the base metal to become molten. Thus, the minimum weld sizes implicitly imply a specified minimum heat input. In addition, restraint to weld metal shrinkage may result in weld cracking if the welds are too small. Minimum weld sizes are frequently used for the case of longitudinal fillet welds that resist shear (e.g., girder flange-to-web welds). Reducing the amount of weld metal will decrease the amount of distortion in welded assemblies; thus, the smallest acceptable weld size that will provide the required factored resistance should be used.

Since the minimum size requirements for fillet welds imply a minimum level of heat input, the minimum size welds must be made in a single pass, as multiple passes to make the minimum size weld would not provide the assumed minimum level of heat input, essentially defeating the purpose of the requirement. The largest single-pass fillet weld that can be made with the manual SMAW process in the horizontal position is typically 5/16 in., but minimum preheat temperature is to be provided. Single-pass fillet welds up to about ½ in. can be made with the SAW process.

The FHWA Bridge Welding Reference Manual is an excellent resource for additional information on the types of welded connections and their design. Extensive guidance on the design of welded connections for steel girder bridges can be found in Section 6.6.4.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. Consult the AASHTO-NSBA Steel Bridge Collaboration Guidelines G1.4-2006

Guidelines for Design Details and G12.1-2020 Guidelines to Design for Constructability and Fabrication for recommendations, commentary, and sample design details that allow for more economical fabrication and erection.

For specific design examples of connections utilizing fillet welds, consult Section 6.6.4.3.7.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.13.3.5 Minimum Effective Length of Fillet Welds

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The provisions of this Article specify the minimum effective length of fillet welds. When placing a fillet weld, the welder builds up the weld to the full dimension as near to the beginning of the weld as possible. However, there is always a slight tapering off of the weld where the weld starts and ends. Therefore, a minimum effective length of the weld is required. As specified in this Article, the minimum effective length of a fillet weld is to be taken as four times its leg size, but not less than 1.5 inches.

The FHWA Bridge Welding Reference Manual is an excellent resource for additional information on the types of welded connections and their design. Extensive guidance on the design of welded connections for steel girder bridges can be found in Section 6.6.4.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. Consult the AASHTO-NSBA Steel Bridge Collaboration Guidelines G1.4-2006 Guidelines for Design Details and G12.1-2020 Guidelines to Design for Constructability and Fabrication for recommendations, commentary, and sample design details that allow for more economical fabrication and erection.

For specific design examples of connections utilizing fillet welds, consult Section 6.6.4.3.7.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.13.3.6 Fillet Weld End Returns

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article deal with end returns on fillet-welded connections.

Fillet welds that resist a tensile force not parallel to the axis of the weld, or are not proportioned to withstand repeated stress, are not to terminate at corners of parts or members. Where such returns can be made in the same plane, they are to be returned continuously, full size, around the corner, for a length equal to twice the weld size and are to be indicated in the contract documents. Also, fillet welds deposited on the opposite sides of a common plane of contact between two parts are to be interrupted at a corner common to both welds.

These situations are not typically encountered in the fillet-welded connections used in the routine steel I-girder bridges covered by this Guide. An example of this condition might be the welded connection of a cross-frame member to a gusset plate if it were proposed to only provide a weld along the end of the member; such a weld would be prohibited by this Article unless returns were provided, effectively resulting in a three-sided connection (the end weld, then the weld wrapped around to the sides of the member). However, such a detail would likely be subject to numerous other design problems; a more robust three-sided welded connection with longer welds along the sides of the member would be a better approach and is in fact the much more common detail used in this situation.

The FHWA Bridge Welding Reference Manual is an excellent resource for additional information on the types of welded connections and their design. Extensive guidance on the design of welded connections for steel girder bridges can be found in Section 6.6.4.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS. Consult the AASHTO-NSBA Steel Bridge Collaboration Guidelines G1.4-2006 Guidelines for Design Details and G12.1-2020 Guidelines to Design for Constructability and Fabrication for recommendations, commentary, and sample design details that allow for more economical fabrication and erection.

6.13.3.7 Fillet Welds for Sealing

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article cover fillet welds to be used for sealing of connections to help prevent penetration of moisture.

Seal welds should be continuous welds combining the functions of sealing and strength. The portion of a return sealing fillet weld wrapped around the ends of a transverse, longitudinal or bearing stiffener, connection plate, or a lap splice connection, if permitted by the Owner-agency, is to be exempt from the minimum size requirements specified in Article 6.13.3.4 (see the Discussion of Article 6.13.3.4 in this Guide), and is not to be considered in determining the resistance of the connection. Research has shown that when wrapping the fillet weld around the ends of stiffeners or connection plates, the undercutting of the corner of the stiffener or connection plate that occurs, even when severe, does not reduce the fatigue performance of the weld, which is controlled by the toe of the transverse fillet weld connecting the stiffener or connection plate to the flange.

As described in the Commentary for this Article, the ends of fillet-welded connections in galvanized structures, in particular, should be sealed to prevent the acids used to prepare the steel for galvanizing from being trapped in-between the components and then leaching out. Vent holes or an unwelded length around the adjoining surfaces should be provided to allow the trapped air and moisture to escape and prevent destructive pressures from developing between the surfaces if the overlapped area is greater than or equal to 16.0 in.^{2} for plates 0.5 in. or less in thickness or greater than or equal to 64.0 in.^{2} for plates greater than 0.5 in. in thickness. Tables 1 and 2 in the ASTM A385 specification provide further guidance along with the size of the vent holes or the unwelded length that is required when the overlapped area exceeds the preceding values..

The FHWA Bridge Welding Reference Manual is an excellent resource for additional information on the types of welded connections and their design. Extensive guidance on the design of welded connections for steel girder bridges can be found in Section 6.6.4.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS. Consult the AASHTO-NSBA Steel Bridge Collaboration Guidelines G1.4-2006 Guidelines for Design Details and G12.1-2020 Guidelines to Design for Constructability and Fabrication for recommendations, commentary, and sample design details that allow for more economical fabrication and erection.

6.13.4 Block Shear Rupture Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

A connected element subject to tension must be checked for a tearing limit state known as block shear rupture. This Article specifies that block shear rupture is to be checked for the web connection of coped beams and for tension connections, including connection plates, splice plates and gusset plates. Tests on coped beams have indicated that a block shear failure can occur around the perimeter of the bolt holes. Tension member connections are also susceptible, and the block shear rupture mode should also be checked around the periphery of welded connections.

The factored tensile resistance of the member, P_r (see the Discussion of Article 6.8.2.1 in this Guide), must not exceed the factored block shear rupture resistance, R_r, of the connected elements, which must be investigated at the end connections. The connections are to be investigated by considering all possible failure planes in the connected elements, including those parallel and perpendicular to the applied forces, and determining the most critical set of planes. For a bolted connection, the failure path is defined by the centerlines of the bolt holes. Planes parallel to the applied force are to be considered to resist only shear stresses and planes perpendicular to the applied force are to be considered to resist only tensile stresses.

The factored block shear rupture resistance, R_r, is determined from Eq. 6.13.4-1. In determining R_r, the resistance to rupture along the shear plane is added to the resistance to rupture on the tensile plane. Block shear rupture is a rupture or tearing phenomenon and not a yielding phenomenon. However, gross yielding along the shear place can occur when tearing on the tensile plane commences if $0.58F_uA_{vn}$ exceeds $0.58F_yA_{vg}$. Therefore, Eq. 6.13.4-1 limits $0.58F_uA_{vn}$ to not exceed $0.58F_yA_{vg}$.

The reduction factor, U_{bs}, has been included in Eq. 6.13.4-1 to approximate the effect of a non-uniform stress distribution on the tensile plane in certain cases; e.g., coped beam connections with multiple rows of bolts. In such cases, the tensile stress on the end plane is non-uniform because the rows of bolts nearest the beam end pick up most of the shear. For the majority of connections encountered in the routine steel I-girder bridges covered by this Guide, U_{bs} will equal 1.0.

The reduction factor, R_p, in Eq. 6.13.4-1 conservatively accounts for the reduced rupture resistance in the vicinity of bolt holes punched full size (see the Discussion of Article 6.8.2.1 in this Guide). Article 6.6.1.2.3 specifies that unless information is available to the contrary, bolt holes in bracing members and their connection plates are to be assumed for design to be punched full size (see the Discussion of Article 6.6.1.2.3 in this Guide). Bracing member connections are often punched full size, whereas bolt holes in splice connections are typically drilled full size, but unless this is explicitly directed in the Owner-agency's specifications, the Engineer should assume the holes are punched full size.

In determining the net area of cuts carrying shear stress in bolted connections with staggered holes, the full effective diameter of the staggered holes centered within two hole diameters of the cut is to be deducted; holes further removed are to be disregarded. In determining the net area of cuts carrying tension stress, the effect of staggered holes adjacent to the cuts is to be determined using the $s^2/4g$ correction described in Article 6.8.3 (see the Discussion of Article 6.8.3 in this Guide).

Block shear rupture is most likely to control in the design of bolted end connections to thin webs of girders (e.g., coped beams) and in the design of short compact bolted connections. Although it must also be checked for splice connections, it is unlikely to control in the design of bolted flange and web splices of typical proportions (see the Discussion of Article 6.13.6 in this Guide).

For further information on the block shear rupture resistance and design examples illustrating the block shear rupture resistance checks, consult Sections 6.6.3.3.2.5 and 6.6.4.2.5.6.1 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.13.5 Connection Elements

6.13.5.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

This Article indicates that the provisions of Article 6.13.5 are to be applied to the design of connection elements such as splice plates, gusset plates, corner angles, brackets, and lateral connection plates in tension or shear, as applicable. For the routine steel I-girder bridges covered by this Guide, these provisions are to be applied to the design of splice plates and cross-frame gusset plates only, as applicable.

6.13.5.2 Tension

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article specifies that the factored tensile resistance, R_r, of a connected element is to be taken as the smallest of the resistances based on yielding, net section fracture or block shear rupture. For the routine steel I-girder bridges covered by this Guide, these provisions are to be applied to determine the factored tensile resistance of flange splice plates and cross-frame gusset plates, as applicable.

A connected element subject to tension must be checked for yielding on the gross section. Excessive elongation due to uncontrolled yielding of the gross area can limit the structural usefulness of the connected element so that it no longer serves its intended purpose. The factored yield resistance, R_r, of a connected element in tension is to be computed from Eq. 6.8.2.1-1 (see the Discussion of Article 6.8.2.1 in this Guide).

A connected element subject to tension must also be checked for fracture on the net section. The connected element can fracture by failure of the net area at a load smaller than that required to yield the gross area depending on the ratio of net to gross area, the properties of the steel (i.e., the ratio of F_u/F_y), and the end connection geometry. Holes in a member cause stress concentrations at service loads, with the tensile stress adjacent to the hole typically about three times the average stress on the net area. As the load increases and the deformation continues, all fibers across the section will achieve or eventually exceed the yield strain. Failure occurs when the localized yielding results in a fracture through the net area. Typically, a higher margin of safety is used when considering the net section fracture resistance versus the yield resistance. The factored net section fracture resistance, R_r, of a connected element in tension is to be computed from Eq. 6.8.2.1-2 (see the Discussion of Article 6.8.2.1 in this Guide).

The calculation of the net area, A_n, in Eq. 6.8.2.1-2 is discussed in Article 6.8.3 (see the Discussion of Article 6.8.3 in this Guide). According to the provisions of this Article, for flange splice plates and cross-frame gusset plates, A_n is not to be taken greater than 85 percent of the gross area, A_g, of the plate in checking Eq. 6.8.2.1-2. Should A_n equal or exceed $0.85A_g$, then $0.85A_g$ is substituted for A_n when checking Eq. 6.8.2.1-2. Because the length of these elements is small compared to the overall member length, inelastic deformation of the gross area is limited. Tests have shown that when holes are present in such short elements where general yielding on the gross section cannot occur, there will be at least a 15 percent reduction in tensile capacity from that obtained based on yielding of the gross section.

The reduction factor, *U*, in Eq. 6.8.2.1-2 accounts for the effect of shear lag in connections (see the Discussion of Article 6.8.2.2 in this Guide). According to the provisions of this Article, for short connection elements such as flange splice plates and cross-frame gusset plates, where the elements of the cross-section essentially lie in a common plane and are connected by bolts or welds, *U* is to be taken equal to 1.0 (except for the rare case where Case 4 in Table 6.8.2.2-1 applies; i.e., when gusset plates are attached to cross-frame connection plates using only longitudinal welds along the length of the connection with no transverse weld across the end of the connection – see the Discussion of Article 6.8.2.2 in this Guide).

The reduction factor, R_p, in Eq. 6.8.2.1-2 conservatively accounts for the reduced rupture resistance in the vicinity of bolt holes punched full size (see the Discussion of Article 6.8.2.1 in this Guide). Article 6.6.1.2.3 specifies that unless information is available to the contrary, bolt holes in connection plates are to be assumed for design to be punched full size (see the Discussion of Article 6.6.1.2.3 in this Guide). Bolt holes in flange splice plates are typically drilled full size, whereas bolt holes in cross-frame gusset plates are often punched full size, but unless this is explicitly directed in the Owner-agency's specifications, the Engineer should assume the holes are punched full size.

Lastly, a connected element subject to tension must also be checked for block shear rupture. The factored block shear rupture resistance, R_r, of the connected element is calculated according to Eq. 6.13.4-1 (see the Discussion of Article 6.13.4 in this Guide).

For further information on the factored tensile resistance of connection elements and design examples illustrating the factored tensile resistance checks, consult Section 6.6.4.2.5.6.1 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS. NSBA's Bolted Field Splices for Steel Bridge Flexural Members – Overview and Design Examples also provides illustrations of these calculations in the context of bolted field splice design.

6.13.5.3 Shear

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article specifies that the factored shear resistance, R_r, of a connected element is to be taken as the smallest of the resistances based on shear yielding or shear rupture. For the routine steel I-girder bridges covered by this Guide, these provisions are to be applied to determine the factored shear resistance of web splice plates and cross-frame gusset plates, as applicable.

For shear yielding, the factored shear resistance of the connected element, R_r, given by Eq. 6.13.5.3-1 is conservatively based on the shear yield stress (i.e., $F_y/\sqrt{3} = 0.58F_y$).

For shear rupture, the factored shear resistance of the connected element, R_r, is given by Eq. 6.13.5.3-2. A higher margin of safety is used when considering the shear rupture resistance versus the shear yield resistance.

The reduction factor, R_p, in Eq. 6.13.5.3-2 conservatively accounts for the reduced rupture resistance in the vicinity of bolt holes punched full size (see the Discussion of Article 6.8.2.1 in this Guide). Article 6.6.1.2.3 specifies that unless information is available to the contrary, bolt holes in connection plates are to be assumed for design to be punched full size (see the Discussion of Article 6.6.1.2.3 in this Guide). Bolt holes in web splice plates are typically drilled full size, whereas bolt holes in cross-frame gusset plates are often punched full size, but unless this is explicitly directed in the Owner-agency's specifications, the Engineer should assume the holes are punched full size.

For further information on the factored shear resistance of connection elements and design examples illustrating the factored shear resistance checks, consult Section 6.6.4.2.5.6.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.13.6 Splices

6.13.6.1 Bolted Splices

6.13.6.1.1 Tension Members

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

A splice is defined as a group of bolted connections (or a welded connection) sufficient to transfer the moment, shear, axial force or torque between two structural elements joined at their ends to form a single, longer element. Bolted splices are typically used to connect member sections together in the field; hence, the term "field splice" is often used. The provisions of this Article cover the design of bolted splices for members subject to axial tension.

Bolted splices for tension members are to be designed using slip-critical connections (see the Discussion of Article 6.13.2.1.1 in this Guide) and are to satisfy the tensile resistance requirements for connected elements specified in Article 6.13.5.2 (see the Discussion of Article 6.13.5.2 in this Guide). The splices are to be designed at the strength limit state for the load as determined by the requirements of Article 6.13.1 for splices of primary members subject to axial tension (see the Discussion of Article 6.13.1 in this Guide). Where the section changes at the splice, the smaller of the two connected sections is to be used in the design.

These provisions are not applicable to the routine steel I-girder bridges covered by this Guide as the members in these bridges which may be subject to axial tension (such as cross-frame members) are not typically spliced, nor should they be.

Provisions for the design of bolted field splices are addressed in Article 6.13.6.1.3, Flexural Members, and its associated sub-Articles.

For further information on field splice design, consult Section 6.6.5 and especially 6.6.5.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS. For in-depth information on the current AASHTO provisions for the design of bolted field splices for flexural members and examples illustrating the design of such splices, consult NSBA's Bolted Field Splices for Steel Bridge Flexural Members – Overview and Design Examples.

6.13.6.1.2 Compression Members

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

A splice is defined as a group of bolted connections (or a welded connection) sufficient to transfer the moment, shear, axial force or torque between two structural elements joined at their ends to form a single, longer element. Bolted splices are typically used to connect member sections together in the field; hence, the term "field splice" is often used. The provisions of this Article cover the design of bolted splices for members subject to axial compression (e.g., arch members, truss chords, and columns).

Splices for compression members may either be designed at the strength limit state as: 1) open joints (i.e., no contact between adjoining parts) with enough bolts provided in the splice to carry 100 percent of the load as determined by the requirements of Article 6.13.1 for splices of primary members subject to axial compression (see the Discussion of Article 6.13.1 in this Guide), or 2) milled joints in full contact bearing with the bolts designed to carry no less than 50 percent of the lower factored resistance of the sections spliced. If the latter option is chosen, Article 6.13.6.1.2 requires that the contract documents call for inspection of the joint during fabrication and erection. Fabricators generally prefer the first option because it is less expensive and has the potential for fewer problems in the field.

The splices in these members are to be located as near as practicable to the panel points and usually on the side of the panel point where the smaller force effect occurs. The arrangement of splice elements must make provision for the various force effects in the component parts of the spliced members.

These provisions are not applicable to the routine steel I-girder bridges covered by this Guide as the members in these bridges which may be subject to axial compression (such as cross-frame members) are not typically spliced, nor should they be.

Provisions for the design of bolted field splices are addressed in Article 6.13.6.1.3, Flexural Members, and its associated sub-Articles.

For further information on field splice design, consult Section 6.6.5 and especially 6.6.5.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS. For in-depth information on the current AASHTO provisions for the design of bolted field splices for flexural members and examples illustrating the design of such splices, consult NSBA's Bolted Field Splices for Steel Bridge Flexural Members – Overview and Design Examples.

6.13.6.1.3 Flexural Members

6.13.6.1.3a General

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

A splice is defined as a group of bolted connections (or a welded connection) sufficient to transfer the moment, shear, axial force or torque between two structural elements joined at their ends to form a single, longer element. Bolted splices are typically used to connect member sections together in the field; hence, the term "field splice" is often used. The provisions of this Article cover general provisions for the design of bolted field splices for members subject to flexure, and hence, are applicable to the routine steel I-girder bridges covered by this Guide.

Bolted beam or girder field splices generally include top flange splice plates, web splice plates and bottom flange splice plates. In addition, if the plate thicknesses on one side of the joint are different than those on the other side, filler plates are used to match the thicknesses within the splice (see the Discussion of Article 6.13.6.1.4 in this Guide). For the flange splice plates, there is typically one plate on the outside of the flange and two smaller plates on the inside of the flange; one on each side of the web. For the web splice plates, there are two plates; one on each side of the web, with at least two rows of high-strength bolts over the depth of the web used to connect the splice plates to the member.

As required by Articles 6.13.6.1.3b and 6.13.6.1.3c, bolted flange and web splice connections are designed at a minimum for 100 percent of the individual design resistances of the flange and web; that is, the individual flange splices are designed for the smaller design yield resistance of the corresponding flanges on either side of the splice (see the Discussion of Article 6.13.6.1.3b in this Guide), and the web splice is designed for the smaller factored shear resistance of the web on either side of the splice (see the Discussion of Article 6.13.6.1.3c in this Guide). Additional forces in the web connection may need to be considered if the flanges are not adequate to develop the factored design moment at the point of splice.

Bolted splices in continuous spans should be made in regions of lower moment at or near points of permanent load contraflexure (inflection points) to reduce the size of the splice. This may not always be possible in certain situations, such as in longer-span bridges or in cases where additional field splices may be needed to reduce the size of a shipping piece to practical limits to better accommodate shipping (e.g., shipping of a sharply curved member).

In cases where bolted splices are located away from points of permanent load contraflexure, the Engineer should check the girder flanges subject to tension for net section fracture since the flanges will have holes for the splice plate bolts. Eq. 6.10.1.8-1 provides a limit on the maximum factored major-axis bending stress permitted on the gross section of the girder, neglecting the loss of area due to holes in the tension flange at the bolted splice (see the Discussion of Article 6.10.1.8 in this Guide). Eq. 6.10.1.8-1 will prevent a bolted splice from being located at a section where the factored flexural resistance of the section at the strength limit state exceeds the moment at first yield, M_y, unless the factored stress in the tension flange at that section is limited to the value given by the equation.

Splices located in areas of stress reversal near points of permanent load contraflexure are to be investigated for both positive and negative flexure to determine the governing condition. Web and flange splices must have at least two rows of bolts on each side of the joint to facilitate proper alignment and stability of the girder during construction. Also, oversize or slotted holes are not to be used in either the member or the splice plates at bolted splices to provide geometry control during erection before the bolts are tightened. Bolted splice connections for flexural members are to be designed as slip-critical connections (see the Discussion of Article 6.13.2.1.1 in this Guide). Bolted connections for flange and web splices are to be proportioned to prevent slip under Load Combination Service II and during the casting of the concrete deck to provide geometry control.

The nominal fatigue resistance of base metal at the gross section adjacent to slip-critical bolted connections is based on fatigue detail Category B assuming the bolts are installed in holes drilled full size or subpunched and reamed to size (Table 6.6.1.2.3-1 – Condition 2.1), which is required for bolted beam or girder splices. However, fatigue will not control the design of the bolted splice plates for flexural members. The combined areas of the flange and web splice plates typically will exceed the areas of the smaller flanges and web to which they are attached, and the flanges and web are usually checked separately for either equivalent or more critical fatigue category details. Therefore, fatigue of the splice plates will not need to be checked.

For further information on field splice design, consult Section 6.6.5 and especially 6.6.5.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.. The provisions addressing the design of bolted field splices changed significantly in 2017 with the publication of the 8th Edition of the AASHTO LRFD BDS. For in-depth information on the current AASHTO provisions for the design of bolted field splices for flexural members and examples illustrating the design of such splices, consult NSBA's Bolted Field Splices for Steel Bridge Flexural Members – Overview and Design Examples.

In addition, NSBA's NSBA Splice Microsoft Excel-based bolted field splice design spreadsheet takes the time-consuming task of designing and checking a bolted splice connection and rewrites the process with a simple input page and output form. NSBA Splice can be incorporated as a design tool on plate girder bridges allowing the designer to quickly analyze various bolted splice connections to determine the most efficient bolt quantity and configuration. NSBA Splice allows the user to explore the effects of bolt spacing, bolt size, strength, and connection dimensions on the overall splice design.

6.13.6.1.3b Flange Splices

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The provisions of this Article cover the design of flange splice plates and their connections for bolted field splices in flexural members and are applicable to the routine steel I-girder bridges covered by this Guide.

At the strength limit state, each flange splice must develop the smaller full design yield resistance of the flanges, P_{fy}, on either side of the splice. P_{fy} of each flange based on the effective area of the flange, A_e, is calculated using Eq. 6.13.6.1.3b-1. A_e is calculated using Eq. 6.13.6.1.3b-2 and cannot exceed the gross area of the flange, A_g. In most cases, A_e will be less than A_g. A_e is used rather than A_g to account for the loss in section causing a reduction in the fracture resistance of the net section at the connection for loading conditions in which the flange is subject to tension; that is, the flange splices are designed for a lower design force such that net section fracture will not occur at the connection. A_e only applies to tension flanges but is conservatively applied to both flanges in the design method since each flange may potentially be subject to tension and the resulting design force will conservatively provide a lower moment resistance of the flanges at the strength limit state, as described further below.

The moment resistance provided by the flanges (neglecting the web and considering only the flange force) is next checked against the factored moment at the strength limit state at the point of splice. Should the factored moment exceed the moment resistance provided by the flanges, the additional moment is to be resisted by the web (see the Discussion of Article 6.13.6.1.3c in this Guide). For composite sections subject to positive flexure, the moment resistance provided by the flanges is computed as shown in Figure C6.13.6.1.3b-1. For composite sections subject to negative flexure and for noncomposite sections subject to positive or negative flexure, the moment resistance provided by the flanges is computed as shown in Figure C6.13.6.1.3b-2. The moment resistance provided by the flanges can potentially be increased by staggering the flange bolts.

As discussed in the Commentary to this Article, for composite sections where: 1) shear connectors are provided in regions of negative flexure such that the longitudinal reinforcement may be considered part of the composite section, as permitted in Article 6.10.10.1 (see the Discussion of Article 6.10.10.1 in this Guide); and 2) the longitudinal reinforcement is considered in the section properties for calculating flexural stresses acting on composite sections subject to negative flexure as specified in Article 6.10.1.1.1c (see the Discussion of Article 6.10.1.1.1c in this Guide), or for computing the factored moment resistance of the composite section in negative flexure, as applicable, the moment resistance provided by the flanges at the strength limit state at the point of splice for composite sections subject to negative flexure can potentially be increased if desired. Both of the preceding conditions are recommended for the routine steel I-girder bridges covered in this Guide. This increase, which will allow the design of the bolted splice to be compatible with the moment design of the section when the longitudinal reinforcement is included in the moment design of the section, may be accomplished by including the additional moment resistance provided by the design yield resistance of the longitudinal deck reinforcement, P_{rs}, which is equal to $A_{rs}F_{yr}$, where A_{rs} is the total area of the longitudinal reinforcement included in the composite section satisfying the size, spacing, and placement requirements specified in Article 6.10.1.7 (see the Discussion of Article 6.10.1.7 in this Guide), but not to exceed one percent of the total cross-sectional area of the concrete deck assumed distributed within the appropriate effective width of the concrete deck acting with the girder under consideration. F_{yr} is the specified minimum yield strength of the longitudinal reinforcement. The Commentary further goes on to describe how to compute the moment resistance including P_{rs}.

The number of bolts required on one side of the flange splice at the strength limit state is found by dividing P_{fy} by the factored shear resistance of the bolts (see the Discussion of Articles 6.13.2.2 and 6.13.2.7 in this Guide), including the reduction in the shear resistance of the bolts due to any needed fillers (see the Discussion of Article 6.13.6.1.4 in this Guide). The threads typically are excluded from the shear plane in flange splices (see the Discussion of Article 6.13.2.7 in this Guide for further discussion of evaluating when threads may be assumed to be excluded from the shear plane). The number of shear planes, N_s, is determined as described further below. For a bolt in a lap-splice connection greater than 38.0 in. in length, the nominal shear resistance, R_n, is reduced by a factor of 0.83 (see the Discussion of Article 6.13.2.7 in this Guide). For bolted flange splices, the 38.0 in. length is measured between the extreme bolts on only one side of the splice and is normally not exceeded.

The bearing resistance of the flange splice bolt holes is to be checked at the strength limit state (see the Discussion of Article 6.13.2.9 in this Guide). The bearing resistance of the connection is taken as the sum of the smaller of the shear resistance of the individual bolts and the bearing resistance of the individual bolt holes parallel to the line of the design force. If the bearing resistance of a bolt hole exceeds the shear resistance of the bolt, the bolt resistance is limited to the shear resistance. Assuming the sum of the flange splice-plate thicknesses exceeds the thickness of the thinner flange at the point of splice, and the splice plate areas satisfy the 10 percent rule described below, the bearing resistance of the connection will be governed by the flange on either side of the splice with the smaller product of the thickness and specified minimum tensile strength, F_u, of the flange. Otherwise, the bearing resistance of each individual component should be checked to determine the component governing the bearing resistance of the connection.

At the strength limit state, P_{fy} may be assumed equally divided to the inner and outer flange splice plates when the areas of the inner and outer plates do not differ by more than 10 percent. In this case, the shear resistance of the bolted connection should be checked for P_{fy} acting in double shear (i.e., $N_s = 2$). Should the areas of the inner and outer splice plates differ by more than 10 percent, the force in each plate should be determined by multiplying P_{fy} by the ratio of the area of the splice plate under consideration to the total area of the inner and outer plates. In this case, the shear resistance of the bolted connection should be checked for the larger of the calculated splice-plate forces acting on a single shear plane (i.e., $N_s = 1$). The width of the outside splice plate should be at least as wide as the width of the narrowest flange at the splice. The thickness of the outside splice plate should be at least one-half the thickness of the thinner flange at the splice plus 1/16 of an inch. The width of the inner splice plates should be chosen to allow a clearance distance of at least 1/8-inch between the edge of each splice plate and the adjacent flange-to-web weld.

The design force in flange splice plates subject to tension at the strength limit state is not to exceed the factored resistance of the splice plates in tension; that is, the splice plates are to be checked for yielding on the gross section, fracture on the net section, and for block shear rupture (see the Discussion of Articles 6.13.5.2 and 6.13.4 in this Guide). Block shear rupture will not typically control the design of flange splice plates of typical proportion. Furthermore, the design net area of the splice plates, A_n, must not exceed $0.85A_g$, where A_g is the gross area of the splice plates. Should A_n equal or exceed $0.85A_g$, then $0.85A_g$ is substituted for A_n when checking fracture on the net section of the splice plates; otherwise, A_n is used. The factors, U, R_p, and U_{bs}, are to be taken equal to 1.0 for splice plates in the net section fracture and block shear rupture checks. The factored yield resistance of the splice plates in compression, R_r, is the same as the factored yield resistance of the splice plates in tension, and therefore, need not be checked. Buckling of the splice plates in compression is not a concern since the unsupported length of the plates is limited by the maximum bolt spacing and end distance requirements (see the Discussion of Articles 6.13.2.6.2 and 6.13.2.6.5 in this Guide).

The moment resistance provided by the nominal slip resistance of the flange splice bolts (see the Discussion of Article 6.13.2.8 in this Guide) is to also be checked against the factored moment for checking slip. The factored moments for checking slip are taken as the moment at the point of splice under Load Combination Service II (see the Discussion of Article 3.4.1 in this Guide), and the factored moment at the point of splice due to the deck casting sequence (see the Discussion of Article 3.4.2.1 in this Guide). When checking the slip resistance of the bolts for a typical flange splice with inner and outer splice plates, the flange slip force is assumed divided equally to the two slip planes regardless of the ratio of the splice plate areas (i.e., $N_s = 2$). Unless slip occurs on both planes, slip of the connection cannot occur. The moment resistance provided by the nominal slip resistance of the flange splice bolts is computed as described above for the strength limit state, with the nominal slip resistance of the bolts substituted for P_{fy}. For checking slip due to the factored deck casting moment, the moment resistance of the noncomposite section is used. Should the flange bolts not be sufficient to resist the factored moment for checking slip, the additional moment is to be resisted by the web (see the Discussion of Article 6.13.6.1.3c in this Guide). Should the web bolts not be sufficient to resist the additional moment, only then should consideration be given to adding additional bolts to the flange splices.

The portions of this Article and the associated Commentary dealing with box sections and horizontally curved girders are not applicable to the routine steel I-girder bridges covered by this Guide.

For further information on field splice design, consult Section 6.6.5 and especially 6.6.5.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS. The provisions addressing the design of bolted field splices changed significantly in 2017 with the publication of the 8th Edition of the AASHTO LRFD BDS. For in-depth information on the current AASHTO provisions for the design of bolted field splices for flexural members and examples illustrating the design of such splices, consult NSBA's Bolted Field Splices for Steel Bridge Flexural Members – Overview and Design Examples.

In addition, NSBA's NSBA Splice Microsoft Excel-based bolted field splice design spreadsheet takes the time-consuming task of designing and checking a bolted splice connection and rewrites the process with a simple input page and output form. NSBA Splice can be incorporated as a design tool on plate girder bridges allowing the designer to quickly analyze various bolted splice connections to determine the most efficient bolt quantity and configuration. NSBA Splice allows the user to explore the effects of bolt spacing, bolt size, strength, and connection dimensions on the overall splice design.

6.13.6.1.3c Web Splices

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The provisions of this Article cover the design of web splice plates and their connections for bolted field splices in flexural members and are applicable to the routine steel I-girder bridges covered by this Guide.

Web splices must be designed at a minimum at the strength limit state for a design web force taken equal to the smaller factored shear resistance of the web, V_r, on either side of the splice (see the Discussion of Article 6.10.9 in this Guide). Since the web splice is being designed to develop the full factored shear resistance of the web at a minimum and the eccentricity of the shear is small relative to the depth of the connection, the effect of the small moment introduced by the eccentricity of the web connection is ignored at all limit states.

If the moment resistance provided by the flanges (see the Discussion of Article 6.13.6.1.3b in this Guide) is not sufficient to resist the factored moment at the strength limit state at the point of splice, the web splice connections are to be designed for a resultant design web force, R, taken equal to the vector sum of the smaller factored shear resistance, V_r, and a horizontal force, H_w, in the web that provides the required moment resistance in conjunction with the flange splices. For composite sections subject to positive flexure, H_w is computed as shown in Figure C6.13.6.1.3c-1. For composite sections subject to negative flexure and for noncomposite sections subject to positive or negative flexure, H_w is computed as shown in Figure C6.13.6.1.3c-2. The web moment in these figures is the portion of the factored moment at the strength limit state at the point of splice that exceeds the moment resistance provided by the flanges. Because the resultant web force is assumed divided equally to all the bolts, the traditional vector analysis for connections subject to eccentric shear is not applied.

For a composite section at a bolted splice located in a higher moment area and subject to positive flexure where the moment resistance provided by the flanges at the point of splice is not sufficient to resist the factored moment at the strength limit state, if the total tensile demand at the point of splice, i.e., the sum of H_w and the design yield resistance of the bottom flange, P_{fy}, exceeds the compressive resistance of the concrete deck, i.e., $0.85f'_c b_{eff} t_s$ where b_{eff} and t_s are the effective width (see the Discussion of Article 4.6.2.6 in this Guide) and thickness of the concrete deck, respectively, H_w should instead be determined as shown in Figure C6.13.6.1.3c-2 for composite sections subject to negative flexure and noncomposite sections. That is, H_w should be determined as the web moment divided by the moment arm taken as one-quarter of the web depth. This conservatively ignores the contribution of the concrete deck and concrete haunch to the moment resistance provided by the web. In the calculation of the web moment for the determination of H_w in this case, the moment resistance provided by the flanges should be computed as shown in Figure C6.13.6.1.3b-1 using the smaller of P_{fy} and the compressive resistance of the concrete deck. Although not currently covered or discussed in the 10th Edition AASHTO LRFD BDS, this is an important design consideration that will likely be included in the next edition of the BDS.

The number of bolts required on one side of the web splice at the strength limit state is found by dividing design web force, V_r or R as applicable, by the factored shear resistance of the bolts (see the Discussion of Articles 6.13.2.2 and 6.13.2.7 in this Guide). The threads typically are included in the shear plane in web splices (see the Discussion of Article 6.13.2.7 in this Guide for further discussion of evaluating when threads may be assumed to be excluded from the shear plane).. The number of shear planes, N_s, is equal to two for web splices. The joint length reduction factor of 0.83 applies only to lap-splice tension or compression connections greater than 38.0 in. in length and does not apply to web splices (see the Discussion of Article 6.13.2.7 in this Guide). At a minimum, two vertical rows of bolts spaced at the maximum spacing for sealing bolts should be provided (see the Discussion of Article 6.13.2.6.2 in this Guide) with a closer spacing and/or additional rows provided only as needed.

The bearing resistance of the web splice bolt holes is to be checked at the strength limit state (see the Discussion of Article 6.13.2.9 in this Guide). The bearing resistance may be calculated as the sum of the smaller of the shear resistance of the individual bolts and the bearing resistance of the individual bolt holes parallel to the line of the design force. If the bearing resistance of a bolt hole exceeds the shear resistance of the bolt, the bolt resistance is limited to the shear resistance. When a moment contribution from the web is required at the strength limit state, the resultant forces causing bearing on the web bolt holes are inclined; that is, with a horizontal component of force equal to H_w divided by the number of web bolts and a vertical component of force equal to V_r divided by the number of web bolts. The bearing resistance of each bolt hole in the web can conservatively be calculated in this case using the clear edge distance, as shown on the left-hand side of Figure 6.13.6.1.3c. This calculation is conservative since the resultant forces act in the direction of inclined distances larger than the clear edge distance. This calculation is also likely to be a conservative calculation for the bolt holes in the adjacent rows. Should the bearing resistance be exceeded, it is recommended that the clear edge distance be increased slightly in lieu of increasing the number of bolts or thickening the web. Other possible options are discussed in the

Commentary for this Article. In rare cases where the bearing resistance is controlled by the web splice plates, the smaller of the clear edge or end distance on the splice plates can conservatively be used to compute the bearing resistances of each hole, as shown on the right-hand side of Figure 6.13.6.1.3c.

Webs are to be spliced symmetrically by plates on each side. The splice plates are to extend as near as practical for the full depth between flanges without impinging on bolt assembly clearances. Recommended bolt assembly clearances are given in Tables 7-15 and 7-16, as applicable, of the AISC *Manual of Steel Construction*. The thickness of each web splice plate should be at least one-half the thickness of the thinner web at the splice plus $^{1}/_{16}$ of an inch and must be equal to or greater than the minimum permitted thickness of $^{5}/_{16}$ in. for structural steel (see the Discussion of Article 6.7.3 in this Guide). For bolted web splices with thickness differences of $^{1}/_{16}$ in. or less, filler plates should not be provided. For a web thickness change of $^{1}/_{8}$ in., use a $^{1}/_{8}$ in. filler plate on one side of the web rather than $^{1}/_{16}$ in. filler plates on each side; filler plates $^{1}/_{16}$ in. or less create difficulties in fabrication and handling. A minimum gap of ½ in. between the girder sections at the splice should be provided to provide drainage and allow for fit-up. The factored shear resistance of the web, V_r, is not to exceed the factored shear resistance of the web splice plates; that is, the smaller value based on shear yielding or shear rupture (see the Discussion of Article 6.13.5.3 in this Guide). V_r is also not to exceed the factored block shear rupture resistance of the web splice plates, which is unlikely to control (see the Discussion of Article 6.13.4 in this Guide). The factors, R_p and U_{bs}, are to be taken equal to 1.0 for splice plates in the shear rupture and block shear rupture checks, respectively.

At a minimum, bolted connections for web splices are to be checked for slip under a web slip force taken equal to the factored shear in the web at the point of splice. The factored shear for checking slip is taken as the shear in the web at the point of splice under Load Combination Service II (see the Discussion of Article 3.4.1 in this Guide), or the factored shear at the point of splice due to the deck casting sequence (see the Discussion of Article 3.4.2.1 in this Guide), whichever governs. Should the flange bolts not be sufficient to resist the factored moment for checking slip at the point of splice (see the Discussion of Article 6.13.6.1.3b in this Guide), the web splice bolts should instead be checked for slip under a resultant web slip force taken equal to the vector sum of the governing factored shear and a horizontal force, H_w, located in the web that provides the necessary slip resistance in conjunction with the flange splices. H_w is computed as the portion of the factored moment for checking slip at the point of splice that exceeds the moment resistance provided by the nominal slip resistance of the flange splice bolts divided by the appropriate moment arm, A_w. For composite sections subject to positive flexure, A_w is computed as shown in Figure C6.13.6.1.3c-1. For composite sections subject to negative flexure and for noncomposite sections subject to positive or negative flexure, A_w is computed as shown in Figure C6.13.6.1.3c-2. The computed web slip force is then divided by the nominal slip resistance of the bolts (see the Discussion of Article 6.13.2.8 in this Guide) to determine the number of web splice bolts required on one side of the splice to resist slip. In cases where the moment resistance provided by the flange splice bolts is sufficient at the strength limit state, but a moment contribution from the web is required to resist slip, the number of flange splice bolts may be increased to increase the moment resistance provided by the nominal slip resistance of the flange splice bolts in order to prevent having to add an additional row of web splice bolts to resist the resultant web slip force.

The portions of this Article and the associated Commentary dealing with box sections are not applicable to the routine steel I-girder bridges covered by this Guide.

For further information on field splice design, consult Section 6.6.5 and especially 6.6.5.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS. The provisions addressing the design of bolted field splices changed significantly in 2017 with the publication of the 8th Edition of the AASHTO LRFD BDS. For in-depth information on the current AASHTO provisions for the design of bolted field splices for flexural members and examples illustrating the design of such splices, consult NSBA's Bolted Field Splices for Steel Bridge Flexural Members – Overview and Design Examples.

In addition, NSBA's NSBA Splice Microsoft Excel-based bolted field splice design spreadsheet takes the time-consuming task of designing and checking a bolted splice connection and rewrites the process with a simple input page and output form. NSBA Splice can be incorporated as a design tool on plate girder bridges allowing the designer to quickly analyze various bolted splice connections to determine the most efficient bolt quantity and configuration. NSBA Splice allows the user to explore the effects of bolt spacing, bolt size, strength, and connection dimensions on the overall splice design.

6.13.6.1.4 Fillers

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article cover the design of fillers in axially loaded connections, including truss gusset plate chord splices and bolted field splices for flexural members. As such, the provisions related to fillers in bolted field splices for flexural members are applicable to the routine steel I-girder bridges covered by this Guide when such fillers occur and their thickness is ¼ inch or greater.

Fillers are typically used on bolted flange splices of flexural members (and sometimes on web splices) when the thicknesses of the adjoining plates at the point of splice are different. At bolted flange splices, it is often advantageous to transition one or more of the flange thicknesses down adjacent to the point of splice, if possible, so as to reduce the required size of the filler plate, or possibly change the width of the flanges and keep the thickness constant in order to eliminate the need for a filler plate altogether. Fillers must be secured by additional bolts such that the fillers are an integral part of the connection at the strength limit state; that is, such that the shear planes are well-defined and that no reduction in the factored shear resistance of the bolts results due to bending of the bolts.

Note that changing the girder web depth at bolted field splices to minimize the thickness of fillers is not recommended. The girder web depth should be held constant at field splices in routine steel I-girder bridges.

These provisions provide two choices of methods for developing fillers 0.25 in. or more in thickness. The choices are to either:

1. Extend the fillers beyond the gusset or splice plate with the filler extension secured by enough additional bolts to distribute the total stress uniformly over the combined section of the member or filler; or

2. In lieu of extending and developing the fillers, reduce the factored shear resistance of the bolts (see the Discussion of Article 6.13.2.7 in this Guide) by the factor, R, given by Eq. 6.13.6.1.4-1.

In general, the second method is more commonly used, and the first method is rarely, if ever, used in routine steel I-girder bridges. R accounts for the reduction in the nominal shear resistance of the bolts due to bending in the bolts and will likely result in having to provide additional bolts in the connection to develop the filler(s). Note that the reduction factor is only to be applied on the side of the connections with the filler(s). For practical reasons, consideration should be given to using the same number of bolts on either side of the splice. Normally, the splice plate widths, filler plate widths, and flange widths will be equal in the splice, and the connected plate area will be smaller than the sum of the splice plate areas, such that the area ratio, γ, in the equation for R may simply be taken as the ratio of the thickness of the filler to the thickness of the connected plate.

Note that fillers 0.25-in. or more in thickness are not to consist of more than two plates, unless approved by the Engineer. As discussed further in the Commentary for this Article, the actual total filler thickness may exceed the total filler thickness shown in the contract documents by up to a maximum of 0.25 in.

These provisions also require that the specified minimum yield strength of fillers 0.25 in. or more in thickness not be less than the larger of 70 percent of the specified minimum yield strength of the connected plate and 36.0 ksi. This provision is primarily applicable to designs utilizing steels with a specified minimum yield strength greater than 50 ksi, in which case there are likely to be thinner filler-plate material availability issues. To control the potential for bolt bending and excessive deformation of the connection in such cases, a lower limit on the specified minimum yield strength of the fillers is specified. The effects of yielding of the fillers and connection deformation are not considered to be significant for connections with fillers less than 0.25 in. in thickness. Note that for connections involving the use of weathering steels, a weathering grade product should be specified for the filler-plate material.

The resistance to slip between the filler and either connected part at the service limit state is comparable to the slip resistance that would exist between the connected parts if the filler were not present. Therefore, for slip-critical connections (see the Discussion of Article 6.13.2.1.1 in this Guide), the factored slip resistance of the bolts (see the Discussion of Article 6.13.2.8 in this Guide) is not to be adjusted for the effect of the fillers.

For further information on field splice design, consult Section 6.6.5 and especially 6.6.5.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS. The provisions addressing the design of bolted field splices changed significantly in 2017 with the publication of the 8th Edition of the AASHTO LRFD BDS. For in-depth information on the current AASHTO provisions for the design of bolted field splices for flexural members and examples illustrating the design of such splices, consult NSBA's Bolted Field Splices for Steel Bridge Flexural Members – Overview and Design Examples.

In addition, NSBA's NSBA Splice Microsoft Excel-based bolted field splice design spreadsheet takes the time-consuming task of designing and checking a bolted splice connection and rewrites the process with a simple input page and output form. NSBA Splice can be incorporated as a design tool on plate girder bridges allowing the designer to quickly analyze various bolted splice connections to determine the most efficient bolt quantity and configuration. NSBA Splice allows the user to explore the effects of bolt spacing, bolt size, strength, and connection dimensions on the overall splice design.

6.13.6.2 Welded Splices

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

A splice is defined as a welded connection (or a group of bolted connections) sufficient to transfer the moment, shear, axial force or torque between two structural elements joined at their ends to form a single, longer element. Welded splices are typically used to connect members and/or their components together in the fabrication shop; hence, the term "shop splice" is often used. Welded field splices are less commonly used than bolted field splices; if used, they should be arranged to minimize overhead welding. The provisions of this Article cover the design of welded splices, which must conform to the requirements of the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*.

Welded splices for tension, and compression, and flexural members are to be designed using complete joint penetration (CJP) groove welds (see the Discussion of Article 6.13.3.2.2a in this Guide). Only the provisions of this Article relevant to flexural members are applicable to the routine steel I-girder bridges covered by this Guide. The use of welded lap splice plates should be avoided at welded splices.

Fatigue should be checked at welded splices subject to an applied net tensile stress (determined as specified in Article 6.6.1.2.1) based on the appropriate fatigue detail category for the splice configuration given in Table 6.6.1.2.3-1 (see the Discussion of Articles 6.6.1.2.1 and 6.6.1.2.3 in this Guide).

Changing flange widths at welded shop splices in plate girders should be avoided if possible; flange widths are best changed at bolted field splices. This facilitates "slabbing" of flanges during fabrication, as explained and illustrated in Section 1.5.2 of the AASHTO-NSBA Steel Bridge Collaboration's Guideline G12.1-2020 Guidelines to Design for Constructability and Fabrication. Should it become necessary to splice material of different widths using welded butt joints, symmetric transitions must be used that conform to one of the details shown in Figure 6.13.2.6.2-1. The transition often starts at the butt splice. However, note that Figure 6.13.2.6.2-1 shows a preferred detail in which the butt splice is located a minimum of 3.0 in. from the transition for greater ease in fitting the run-off tabs. At welded butt splices joining material of different thicknesses, the transition (including the weld) must be ground to a uniform slope between the offset surfaces of not more than 1 in 2.5 and must be indicated as such in the contract documents.

Efficiently locating thickness transitions in plate girder flanges is a matter of plate length availability and the economics of welding and inspecting a splice compared to the cost of extending a thicker plate. A shop-welded butt splice should be introduced in a beam or girder flange when the savings in flange material and plate-length limitation or special circumstances dictate. Table 1.5.4-1 in the AASHTO-NSBA Steel Bridge Collaboration's Guideline G12.1-2020 Guidelines to Design for Constructability and Fabrication shows a weight savings per inch of flange width that may be used to evaluate placement of shop splices. The criteria vary so Fabricators should be consulted whenever possible. In the contract plans or specifications, provide criteria the Fabricator may follow to eliminate shop-welded flange splices by extending the thicker plate, subject to the approval of the Engineer. When evaluating the request, the Engineer should review the percent change in deflections and stresses resulting from the extension of the thicker plate.

The provisions of this Article are applicable to the welded butt splices (and welded field splices if used) in the routine steel plate girder bridges covered by this Guide. Welded butt splices are not typically used on the routine steel rolled beam bridges covered by this Guide and therefore these provisions are not applicable to those bridges unless welded field splices are used.

The AASHTO-NSBA Steel Bridge Collaboration's Guideline G12.1-2020 Guidelines to Design for Constructability and Fabrication provides practical guidance on the design and detailing of flanges and flange shop splices to facilitate economical fabrication.

6.13.7 Rigid Frame Connections

6.13.7.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article deals with the connections for a rigid frame, which is defined as a structure in which the connections maintain the angular relationship between the beam and column members under load. This Article states that rigid frame connections are to be designed to resist the factored moments, shear, and axial forces at the strength limit state.

These provisions for the design of rigid frame connections are not applicable to the routine steel I-girder bridges covered by this Guide.

6.13.7.2 Webs

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article requires that rigid frame connections have sufficient strength and stiffness to resist the factored loading effects under the strength limit state. In some cases, stiffening of the web in the connection regions can be beneficial in meeting this requirement. A rigid frame is defined as a structure in which the connections maintain their angular relationship between the beam and column members under load.

These provisions for the design of the beam or connection web in a rigid frame are not applicable to the routine steel I-girder bridges covered by this Guide.

6.14 PROVISIONS FOR STRUCTURE TYPES

6.14.1 Through-Girder Spans

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article apply to the design of through-girder spans. Through-girder spans are bridges in which the top flanges of the main girders are located above the top of the deck. This type of design is not used for the routine steel I-girder bridges covered by this Guide, and as such the provisions are not applicable to their design.

6.14.2 Trusses

6.14.2.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of Article 6.14.2 and its related sub-Articles are primarily intended for the design of truss bridges, where the truss is the main spanning element. These provisions are not intended for the design of truss-type cross-frames, except for the specific application of the provisions of Article 6.14.2.8 to the design of gusset plates used to connect truss-type cross-frame members to cross-frame connection plates (stiffeners) (see the Discussion of Article 6.14.2.8 in this Guide). As a result, Article 6.14.2 and its related sub-Articles, except for Article 6.14.2.8, are not applicable to the design of the routine steel I-girder bridges covered by this Guide.

6.14.2.2 Truss Members

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of Article 6.14.2 and its related sub-Articles are primarily intended for the design of truss bridges, where the truss is the main spanning element. These provisions are not intended for the design of truss-type cross-frames, except for the specific application of the provisions of Article 6.14.2.8 to the design of gusset plates used to connect truss-type cross-frame members to cross-frame connection plates (stiffeners) (see the Discussion of Article 6.14.2.8 in this Guide). As a result, Article 6.14.2 and its related sub-Articles, except for Article 6.14.2.8, are not applicable to the design of the routine steel I-girder bridges covered by this Guide.

6.14.2.3 Secondary Stresses

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of Article 6.14.2 and its related sub-Articles are primarily intended for the design of truss bridges, where the truss is the main spanning element. These provisions are not intended for the design of truss-type cross-frames, except for the specific application of the provisions of Article 6.14.2.8 to the design of gusset plates used to connect truss-type cross-frame members to cross-frame connection plates (stiffeners) (see the Discussion of Article 6.14.2.8 in this Guide). As a result, Article 6.14.2 and its related sub-Articles, except for Article 6.14.2.8, are not applicable to the design of the routine steel I-girder bridges covered by this Guide.

6.14.2.4 Diaphragms

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of Article 6.14.2 and its related sub-Articles are primarily intended for the design of truss bridges, where the truss is the main spanning element. These provisions are not intended for the design of truss-type cross-frames, except for the specific application of the provisions of Article 6.14.2.8 to the design of gusset plates used to connect truss-type cross-frame members to cross-frame connection plates (stiffeners) (see the Discussion of Article 6.14.2.8 in this Guide). As a result, Article 6.14.2 and its related sub-Articles, except for Article 6.14.2.8, are not applicable to the design of the routine steel I-girder bridges covered by this Guide.

6.14.2.5 Camber

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of Article 6.14.2 and its related sub-Articles are primarily intended for the design of truss bridges, where the truss is the main spanning element. These provisions are not intended for the design of truss-type cross-frames, except for the specific application of the provisions of Article 6.14.2.8 to the design of gusset plates used to connect truss-type cross-frame members to cross-frame connection plates (stiffeners) (see the Discussion of Article 6.14.2.8 in this Guide). As a result, Article 6.14.2 and its related sub-Articles, except for Article 6.14.2.8, are not applicable to the design of the routine steel I-girder bridges covered by this Guide.

6.14.2.6 Working Lines and Gravity Axes

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of Article 6.14.2 and its related sub-Articles are primarily intended for the design of truss bridges, where the truss is the main spanning element. These provisions are not intended for the design of truss-type cross-frames, except for the specific application of the provisions of Article 6.14.2.8 to the design of gusset plates used to connect truss-type cross-frame members to cross-frame connection plates (stiffeners) (see the Discussion of Article 6.14.2.8 in this Guide). As a result, Article 6.14.2 and its related sub-Articles, except for Article 6.14.2.8, are not applicable to the design of the routine steel I-girder bridges covered by this Guide.

6.14.2.7 Portal and Sway Bracing

6.14.2.7.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of Article 6.14.2 and its related sub-Articles are primarily intended for the design of truss bridges, where the truss is the main spanning element. These provisions are not intended for the design of truss-type cross-frames, except for the specific application of the provisions of Article 6.14.2.8 to the design of gusset plates used to connect truss-type cross-frame members to cross-frame connection plates (stiffeners) (see the Discussion of Article 6.14.2.8 in this Guide). As a result, Article 6.14.2 and its related sub-Articles, except for Article 6.14.2.8, are not applicable to the design of the routine steel I-girder bridges covered by this Guide.

6.14.2.7.2 Through-Truss Spans

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of Article 6.14.2 and its related sub-Articles are primarily intended for the design of truss bridges, where the truss is the main spanning element. These provisions are not intended for the design of truss-type cross-frames, except for the specific application of the provisions of Article 6.14.2.8 to the design of gusset plates used to connect truss-type cross-frame members to cross-frame connection plates (stiffeners) (see the Discussion of Article 6.14.2.8 in this Guide). As a result, Article 6.14.2 and its related sub-Articles, except for Article 6.14.2.8, are not applicable to the design of the routine steel I-girder bridges covered by this Guide.

6.14.2.7.3 Deck Truss Spans

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of Article 6.14.2 and its related sub-Articles are primarily intended for the design of truss bridges, where the truss is the main spanning element. These provisions are not intended for the design of truss-type cross-frames, except for the specific application of the provisions of Article 6.14.2.8 to the design of gusset plates used to connect truss-type cross-frame members to cross-frame connection plates (stiffeners) (see the Discussion of Article 6.14.2.8 in this Guide). As a result, Article 6.14.2 and its related sub-Articles, except for Article 6.14.2.8, are not applicable to the design of the routine steel I-girder bridges covered by this Guide.

6.14.2.8 Gusset Plates

6.14.2.8.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article addresses general requirements for the configuration and detailing of gusset plates.

The provisions of Article 6.14.2.8 and its associated sub-Articles address the design of gusset plates; the provisions are primarily intended for use in the design of truss bridges, where the truss is the main spanning element, but can be applied to the design of gusset plates used to connect the members of truss-type cross-frames to cross-frame connection plates (stiffeners).

The provision that fasteners connecting each member be symmetrical with the axis of the member, so far as practicable, is intended for the bolted connection of truss members to gusset plates; in most cases for routine steel I-girder bridges, the members of truss-type cross-frames are welded to the gusset plates. It is not required that the bolted connection of the gusset plate itself to the cross-frame connection plate (stiffener) be symmetrical to any given cross-frame member.

Gusset plates for truss-type cross-frames in routine steel I-girder bridges are not considered “chord splices” and are not multilayered and so the related provisions in the following sub-articles are not applicable.

The remaining requirements of this provision are applicable to the design of gusset plates used in truss-type cross-frames of the routine steel I-girder bridges covered by this Guide.

6.14.2.8.2 Multilayered Gusset and Splice Plates

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Multilayered gusset and splice plates are not used in truss-type cross-frames for routine steel I-girder bridges. The combination of a cross-frame gusset plate and a cross-frame connection plate (stiffener) is not a “multilayered gusset plate.” As a result, this Article is not applicable to the design of the routine steel I-girder bridges covered by this Guide.

6.14.2.8.3 Shear Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article addresses the calculation of the factored shear resistance of gusset plates for truss bridges but should also be applied to the design of gusset plates for truss-type cross-frames in the routine steel I-girder bridges covered by this Guide.

The Article includes a unique equation for the factored shear resistance associated with shear yielding of gusset plates (Eq. 6.14.2.8.3-1). Investigation of shear rupture is also required, using Eq. 6.13.5.3-2 (see the Discussion of Article 6.13.5.3 in this Guide). The factored shear resistance is to be taken as the smaller of the factored shear yielding or shear rupture resistance. The shear yielding resistance is typically calculated using the gross area of the plane adjacent to a row of bolts, while the shear rupture resistance is typically calculated using the net area of the plane through a row of bolts (i.e., the area of the plane minus the area removed by the bolt holes). The plane investigated is a plane parallel to the direction of the applied shear loading. The definition of shear planes is dependent on the specific geometry of the gusset plate and the applied loads; in some cases the load may need to be broken into orthogonal components parallel to the lines of bolts. This Article and its associated Commentary include figures which help illustrate how to define shear planes. Generally, it is necessary to use a to-scale drawing of the gusset plate, attached members, and loadings to help define the controlling shear planes.

A check of block shear rupture resistance is typically also necessary, as defined in Article 6.13.4. The block shear rupture perimeter in this check is typically defined as the perimeter of the welded connection of a cross-frame member to the gusset plate. See the Discussion of Article 6.13.4 in this Guide for more information.

For further information on the block shear rupture resistance and design examples illustrating the block shear rupture resistance checks, consult Sections 6.6.3.3.2.5 and 6.6.4.2.5.6.1 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

6.14.2.8.4 Compressive Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article addresses the calculation of the factored compressive resistance of gusset plates for truss bridges but should also be applied to the design of gusset plates for truss-type cross-frames in the routine steel I-girder bridges covered by this Guide. The resistance equations are based on modified column buckling equations and Whitmore section analysis. This Article and its associated Commentary include figures which help illustrate how to define the Whitmore section.

Generally, it is necessary to use a to-scale drawing of the gusset plate, attached members, and loadings to help define the Whitmore section.

6.14.2.8.5 Tensile Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article addresses the calculation of the factored tensile resistance of gusset plates for truss bridges but should also be applied to the design of gusset plates for truss-type cross-frames in the routine steel I-girder bridges covered by this Guide. The provisions state that the factored tensile resistance be taken as the smallest factored resistance in tension based on yielding, fracture, or block shear rupture determined according to the provisions of Article 6.13.5.2 (see the Discussion of Article 6.13.5.2 in this Guide). When calculating the tensile yielding and net section fracture resistances according to the provisions of Article 6.8.2.1 (see the Discussion of Article 6.8.2.1 in this Guide), the Whitmore section defined in Figure 6.14.2.8.5-1 should be used. Generally, it is necessary to use a to-scale drawing of the gusset plate, attached members, and loadings to help define the Whitmore section.

6.14.2.8.6 Chord Splices

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article addresses the design of chord splices for truss bridges. Chord splices are the splices of truss chords when the truss is the main spanning element of the bridge; this Article is not applicable to the design of the routine steel I-girder bridges covered by this Guide since these types of structures do not use chord splices.

6.14.2.8.7 Edge Slenderness

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

This Article defines a proportioning limit related to the slenderness of the unsupported edge of a gusset plate for truss bridges but should also be applied to the design of gusset plates for truss-type cross-frames in the routine steel I-girder bridges covered by this Guide. The proportioning limit is defined using a simple equation which reflects the material properties and thickness of the gusset plate. The provision requires stiffening of the edge if the proportioning limit is not met. However, gusset plates for truss-type cross-frames in routine steel I-girder bridges should not be stiffened as such stiffening adds considerable fabrication expense; if the proportioning limit is not met, increase the thickness of the gusset plate.

6.14.2.9 Half Through-Trusses

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article apply to the design of half through-trusses. Half through-trusses are bridges in which the top chords of the trusses are located above the top of the deck while the bottom chords are located below the deck. This type of design is not used in the routine steel I-girder bridges covered by this Guide, and as such the provisions are not applicable to their design.

6.14.2.10 Factored Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions included in this Article are intended for use in the design of truss members when the truss is the main spanning element of the bridge. Consequently, this Article is not applicable to the design of the routine steel I-girder bridges covered by this Guide.

6.14.3 Orthotropic Deck Superstructures

6.14.3.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Orthotropic steel decks are bridge decks constructed using stiffened steel plates as the structural system of the deck. By the definitions of this Guide, the routine steel I-girder bridges use composite reinforced concrete decks. Thus, Article 6.14.3 and its associated sub-Articles are not applicable.

6.14.3.2 Decks in Global Compression

6.14.3.2.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Orthotropic steel decks are bridge decks constructed using stiffened steel plates as the structural system of the deck. By the definitions of this Guide, the routine steel I-girder bridges covered by this Guide use composite reinforced concrete decks. Thus, Article 6.14.3 and its associated sub-Articles are not applicable.

6.14.3.2.2 Local Buckling

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Orthotropic steel decks are bridge decks constructed using stiffened steel plates as the structural system of the deck. By the definitions of this Guide, the routine steel I-girder bridges covered by this Guide use composite reinforced concrete decks. Thus, Article 6.14.3 and its associated sub-Articles are not applicable.

6.14.3.2.3 Panel Buckling

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Orthotropic steel decks are bridge decks constructed using stiffened steel plates as the structural system of the deck. By the definitions of this Guide, the routine steel I-girder bridges covered by this Guide use composite reinforced concrete decks. Thus, Article 6.14.3 and its associated sub-Articles are not applicable.

6.14.3.3 Effective Width of Deck

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Orthotropic steel decks are bridge decks constructed using stiffened steel plates as the structural system of the deck. By the definitions of this Guide, the routine steel I-girder bridges covered by this Guide use composite reinforced concrete decks. Thus, Article 6.14.3 and its associated sub-Articles are not applicable.

6.14.3.4 Superposition of Global and Local Effects

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Orthotropic steel decks are bridge decks constructed using stiffened steel plates as the structural system of the deck. By the definitions of this Guide, the routine steel I-girder bridges covered by this Guide use composite reinforced concrete decks. Thus, Article 6.14.3 and its associated sub-Articles are not applicable.

6.14.4 Solid Web Arches

6.14.4.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Solid web arches are bridges where the steel arch is the main spanning element of the bridge. Thus, Article 6.14.4 and its associated sub-Articles are not applicable to the design of the routine steel I-girder bridges covered by this Guide.

6.14.4.2 Web Slenderness

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Solid web arches are bridges where the steel arch is the main spanning element of the bridge. Thus, Article 6.14.4 and its associated sub-Articles are not applicable to the design of the routine steel I-girder bridges covered by this Guide.

6.14.4.3 Moment Amplification

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Solid web arches are bridges where the steel arch is the main spanning element of the bridge. Thus, Article 6.14.4 and its associated sub-Articles are not applicable to the design of the routine steel I-girder bridges covered by this Guide.

6.14.4.4 Nominal Compressive Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Solid web arches are bridges where the steel arch is the main spanning element of the bridge. Thus, Article 6.14.4 and its associated sub-Articles are not applicable to the design of the routine steel I-girder bridges covered by this Guide.

6.14.4.5 Nominal Flexural Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Solid web arches are bridges where the steel arch is the main spanning element of the bridge. Thus, Article 6.14.4 and its associated sub-Articles are not applicable to the design of the routine steel I-girder bridges covered by this Guide.

6.14.4.6 Combined Axial Compression or Tension with Flexural and Torsion

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

Solid web arches are bridges where the steel arch is the main spanning element of the bridge. Thus, Article 6.14.4 and its associated sub-Articles are not applicable to the design of the routine steel I-girder bridges covered by this Guide.

6.15 PILES

6.15.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of Article 6.15 and the associated sub-Articles present the requirements for the structural design of steel piles. Other requirements for the design of piles are presented in Chapter 10 of the AASHTO LRFD BDS. Steel piles take several forms, such as steel H-piles or steel pipe piles. The provisions of Article 6.15 and the associated sub-Articles often defer to other Articles in Chapter 6 which directly address the calculation of resistance to axial compressive loads, flexure, or their combined effects. In some cases, H-pile sections have been used for superstructure elements, but in those cases the design is governed by the appropriate related Articles elsewhere in Chapter 6. The provisions of Article 6.15 and its associated sub-Articles are specifically intended for use in the design of these sections when used as piles in the substructure or foundations of a bridge. As such, this Article addresses design items which are beyond the scope of superstructure design.

6.15.2 Structural Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of Article 6.15 and the associated sub-Articles present the requirements for the structural design of steel piles. Other requirements for the design of piles are presented in Chapter 10 of the AASHTO LRFD BDS. Steel piles take several forms, such as steel H-piles or steel pipe piles. The provisions of Article 6.15 and the associated sub-Articles often defer to other Articles in Chapter 6 which directly address the calculation of resistance to axial compressive loads, flexure, or their combined effects. In some cases, H-pile sections have been used for superstructure elements, but in those cases the design is governed by the appropriate related Articles elsewhere in Chapter 6. The provisions of Article 6.15 and its associated sub-Articles are specifically intended for use in the design of these sections when used as piles in the substructure or foundations of a bridge. As such, this Article addresses design items which are beyond the scope of superstructure design.

6.15.3 Compressive Resistance

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of Article 6.15 and the associated sub-Articles present the requirements for the structural design of steel piles. Other requirements for the design of piles are presented in Chapter 10 of the AASHTO LRFD BDS. Steel piles take several forms, such as steel H-piles or steel pipe piles. The provisions of Article 6.15 and the associated sub-Articles often defer to other Articles in Chapter 6 which directly address the calculation of resistance to axial compressive loads, flexure, or their combined effects. In some cases, H-pile sections have been used for superstructure elements, but in those cases the design is governed by the appropriate related Articles elsewhere in Chapter 6. The provisions of Article 6.15 and its associated sub-Articles are specifically intended for use in the design of these sections when used as piles in the substructure or foundations of a bridge. As such, this Article addresses design items which are beyond the scope of superstructure design.

6.15.3.1 Axial Compression

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of Article 6.15 and the associated sub-Articles present the requirements for the structural design of steel piles. Other requirements for the design of piles are presented in Chapter 10 of the AASHTO LRFD BDS. Steel piles take several forms, such as steel H-piles or steel pipe piles. The provisions of Article 6.15 and the associated sub-Articles often defer to other Articles in Chapter 6 which directly address the calculation of resistance to axial compressive loads, flexure, or their combined effects. In some cases, H-pile sections have been used for superstructure elements, but in those cases the design is governed by the appropriate related Articles elsewhere in Chapter 6. The provisions of Article 6.15 and its associated sub-Articles are specifically intended for use in the design of these sections when used as piles in the substructure or foundations of a bridge. As such, this Article addresses design items which are beyond the scope of superstructure design.

6.15.3.2 Combined Axial Compression and Flexure

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of Article 6.15 and the associated sub-Articles present the requirements for the structural design of steel piles. Other requirements for the design of piles are presented in Chapter 10 of the AASHTO LRFD BDS. Steel piles take several forms, such as steel H-piles or steel pipe piles. The provisions of Article 6.15 and the associated sub-Articles often defer to other Articles in Chapter 6 which directly address the calculation of resistance to axial compressive loads, flexure, or their combined effects. In some cases, H-pile sections have been used for superstructure elements, but in those cases the design is governed by the appropriate related Articles elsewhere in Chapter 6. The provisions of Article 6.15 and its associated sub-Articles are specifically intended for use in the design of these sections when used as piles in the substructure or foundations of a bridge. As such, this Article addresses design items which are beyond the scope of superstructure design.

6.15.3.3 Buckling

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of Article 6.15 and the associated sub-Articles present the requirements for the structural design of steel piles. Other requirements for the design of piles are presented in Chapter 10 of the AASHTO LRFD BDS. Steel piles take several forms, such as steel H-piles or steel pipe piles. The provisions of Article 6.15 and the associated sub-Articles often defer to other Articles in Chapter 6 which directly address the calculation of resistance to axial compressive loads, flexure, or their combined effects. In some cases, H-pile sections have been used for superstructure elements, but in those cases the design is governed by the appropriate related Articles elsewhere in Chapter 6. The provisions of Article 6.15 and its associated sub-Articles are specifically intended for use in the design of these sections when used as piles in the substructure or foundations of a bridge. As such, this Article addresses design items which are beyond the scope of superstructure design.

6.15.4 Maximum Permissible Driving Stresses

Determination of applicability, *All Routine Steel I-girder Bridges*: Beyond scope of superstructure design.

Discussion:

The provisions of Article 6.15 and the associated sub-Articles present the requirements for the structural design of steel piles. Other requirements for the design of piles are presented in Chapter 10 of the AASHTO LRFD BDS. Steel piles take several forms, such as steel H-piles or steel pipe piles. The provisions of Article 6.15 and the associated sub-Articles often defer to other Articles in Chapter 6 which directly address the calculation of resistance to axial compressive loads, flexure, or their combined effects. In some cases, H-pile sections have been used for superstructure elements, but in those cases the design is governed by the appropriate related Articles elsewhere in Chapter 6. The provisions of Article 6.15 and its associated sub-Articles are specifically intended for use in the design of these sections when used as piles in the substructure or foundations of a bridge. As such, this Article addresses design items which are beyond the scope of superstructure design.

6.16 PROVISIONS FOR SEISMIC DESIGN

6.16.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The provisions of this Article apply for the seismic design of steel-girder bridge superstructures at the extreme event limit state.

This Article discusses the need to define a clear seismic load path within the superstructure to transmit the inertia forces to the substructure based on the stiffness characteristics of the concrete deck, cross-frames or diaphragms, and bearings. The flow of the seismic forces is to be accommodated through the affected components and connections of the superstructure within the defined load path. This Article also refers to the minimum support-length requirements at expansion bearings specified in Article 4.7.4.4 (see the Discussion of Article 4.7.4.4 in this Guide).

For the application of the seismic design provisions in the AASHTO LRFD BDS, the routine steel I-girder bridges covered by this Guide are assumed to be located in Seismic Zone 1 (see the Discussion of Article 6.16.3 in this Guide).

6.16.2 Materials

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

For the application of the seismic design provisions in the AASHTO LRFD BDS, the routine steel I-girder bridges covered by this Guide are assumed to be located in Seismic Zone 1 (see the Discussion of Article 6.16.3 in this Guide). As specified in this Article, structural steels within the seismic load path are to satisfy the requirements of Article 6.4.1 (see the Discussion of Article 6.4.1 in this Guide). Capacity design to protect members and connections is not to be used for bridges located in Seismic Zone 1; thus, the expected yield strengths defined in this Article should not be used.

6.16.3 Design Requirements for Seismic Zone 1

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

For the application of the seismic design provisions in the AASHTO LRFD BDS, the routine steel I-girder bridges covered by this Guide are assumed to be located in Seismic Zone 1. In addition to the minimum support-length requirements at expansion bearings specified in Article 4.7.4.4 for bridges located in Seismic Zone 1 (see the Discussion of Article 4.7.4.4 in this Guide), the horizontal inertial forces to be transmitted through the restrained bearings are to be calculated based on a specified factor times the vertical reaction due to the tributary permanent load defined in this Article and the tributary live loads assumed to exist during an earthquake, as described further in Article 3.10.9.2 (see the Discussion of Article 3.10.9.2 in this Guide). The magnitude of live load assumed to exist at the time of the earthquake should be consistent with the value of γ_{eq} used in conjunction with Table 3.4.1-1 (see the Discussion of Article 3.4.1 in this Guide). These requirements are intended to provide a clear load path for the seismic forces. Note that the design forces discussed in Article 3.10.9.2 are only for the “connection” of the superstructure to the substructure, and the definition of “connections” (as discussed in the Commentary for Article 3.10.7.1) includes only fixed bearings, expansion bearings with either restrainers, STUs, or dampers, and shear keys. There is no need to apply these forces to the design of pier or end cross-frames or other elements of the superstructure.

The primary components of a routine bridge are intended to have sufficient structural capacity from the design due to satisfaction of the nonseismic design requirements to resist the expected lateral seismic forces in Seismic Zone 1. However, sufficient integrity and structural connectivity still needs to be present to resist the lateral loads at restrained bearings defined in Article 3.10.9.2 in order to mobilize the lateral resistance of the main substructure elements.

Further discussion of the impacts of seismic loading on bearing design and substructure design is considered beyond the scope of the discussion of superstructure design topics in this Guide.

6.16.4 Design Requirements for Seismic Zones 2, 3, or 4

6.16.4.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article define two potential response strategies for the seismic design of bridges located in Seismic Zones 2, 3, or 4: 1) Type A – design an elastic superstructure with a ductile substructure; or 2) Type B – design an elastic superstructure and substructure with a fusing mechanism, e.g. a seismic isolation device, at the interface with the superstructure and substructure (requires Owner-agency approval). Each of these strategies is discussed further in the Commentary for this Article. The use of one of these strategies is required for bridges located in Seismic Zones 3 or 4 and should be considered for bridges located in Seismic Zone 2. Support cross-frame members on bridges located in Seismic Zone 3 or 4 are to be considered primary members for seismic design.

The routine steel I-girder bridges covered by this Guide are assumed to be located in Seismic Zone 1; therefore, the provisions of this Article are not applicable.

6.16.4.2 Deck

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article contains provisions for the design of the concrete deck for bridges located in Seismic Zones 2, 3, or 4 to check that the deck can serve as a horizontal diaphragm to transfer the seismic forces to the supports. Provisions are provided to calculate the transverse seismic shear force acting on the deck within each span of the superstructure for designs using seismic response Strategy Type A or Type B defined in Article 6.16.4.1 (see the Discussion of Article 6.16.4.1 in this Guide).

The routine steel I-girder bridges covered by this Guide are assumed to be located in Seismic Zone 1; therefore, the provisions of this Article are not applicable.

6.16.4.3 Shear Connectors

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

This Article contains provisions for the design of stud shear connectors along the interface between the deck and the steel girders or along the interface between the deck and the top of the support cross-frames or diaphragms, or both, to transfer the seismic forces in bridges located in Seismic Zones 2, 3, or 4. At support locations, shear connectors are to be designed to resist the combination of shear and axial forces corresponding to the transverse seismic shear force within the deck determined according to the provisions of Article 6.16.4.2 (see the Discussion of Article 6.16.4.2 in this Guide). A tension-shear interaction equation is provided in this Article to check the resistance of the stud shear connectors to the combined shear and axial forces.

The routine steel I-girder bridges covered by this Guide are assumed to be located in Seismic Zone 1; therefore, the provisions of this Article are not applicable.

6.16.4.4 Elastic Superstructures

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

In order to achieve an elastic superstructure, this Article specifies that the support cross-frame members or support diaphragms in a bridge located in Seismic Zone 2, 3, or 4 are to be designed according to the applicable provisions of Article 6.7, 6.8, or 6.9 to remain elastic during a seismic event. No other special seismic requirements are specified for these members. The support cross-frame members or diaphragms are to be designed for the lateral force, F, specified in Article 6.16.4.2 for bridges designed using seismic response strategy Type A or Type B, as applicable (see the Discussion of Article 6.16.4.2 in this Guide).

The routine steel I-girder bridges covered by this Guide are assumed to be located in Seismic Zone 1; therefore, the provisions of this Article are not applicable.

6.17 REFERENCES

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article provides reference citations for specifications, reports, and published papers containing more in-depth discussion of the various provisions in Section 6, including the research or reasoning behind the development of the provisions. These useful references are cited in the Commentary for the individual provisions, which reinforces the benefit and importance of reading the Commentary. Not all references in this Article are applicable to the routine steel I-girder bridges covered by this Guide.

APPENDIX A6 FLEXURAL RESISTANCE OF COMPOSITE I-SECTIONS IN NEGATIVE FLEXURE AND NONCOMPOSITE I-SECTIONS WITH COMPACT OR NONCOMPACT WEBS IN STRAIGHT BRIDGES

A6.1 GENERAL

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

The provisions of Appendix A6 account for the ability of certain compact and noncompact web I-sections to develop flexural resistances significantly greater than the yield moment, M_y.

If permitted by the Owner-agency, the optional provisions of Appendix A6 may be applied for the design of composite I-sections in regions of negative flexure and noncomposite I-sections in regions of positive or negative flexure in straight bridges without holes in the tension flange whose supports are normal or skewed not more than 20° from normal and with intermediate cross-frame/diaphragms placed in contiguous lines parallel to the supports, for which: 1) the specified minimum yield strengths of the flanges do not exceed 70 ksi; 2) the web satisfies the noncompact web slenderness limit, λ_{rw}, given by Eqs. A6.1-1 and A6.1-3; and 3) the flanges satisfy the moment-of-inertia ratio given by Eq. A6.1-2. Note that in Eq. A6.1-2, the moments of inertia of the compression and tension flange of the steel section are both taken about the vertical axis in the plane of the web (or about the strong axis of each flange). Sections not satisfying one or more of the above requirements must instead be proportioned according to the provisions of Article 6.10.8 (see the Discussion of Article 6.10.8 in this Guide). The reasoning behind the above limitations on the use of Appendix A6 is described further in the Commentary for this Article and in the Commentary for Articles 6.10.1.8 and 6.10.6.2.3. Since the types of sections that would qualify for the use of Appendix A6 are less commonly used, the somewhat more complex provisions for their design have been placed in an appendix in the AASHTO LRFD BDS to streamline and simplify the provisions of Article 6.10.8 within the main body of the Specifications.

Sections designed according to the provisions of Appendix A6 (i.e., nonslender web sections) are further categorized as either compact web or noncompact web I-sections (see the Discussion of Articles A6.2.1 and A6.2.2 in this Guide).

As stated above, the provisions of Appendix A6 account for the ability of certain compact and noncompact web I-sections to develop flexural resistances significantly greater than the yield moment, M_y (see the Discussion of Article D6.2 in this Guide). As a result, the equations giving the nominal flexural resistance in Appendix A6 are more appropriately expressed in terms of bending moment for reasons discussed in the Commentary for Article 6.10.6.1. The provisions of Article 6.10.8 assume that the section is a slender web section regardless of whether it is or not; hence, the nominal flexural resistance computed according to the provisions of Article 6.10.8 is not permitted to exceed M_y. The provisions of Appendix A6 also account for the contribution of the St. Venant torsional resistance to the lateral-torsional buckling resistance of these sections, which may be useful for compact and noncompact web sections with larger unbraced lengths, particularly under construction conditions.

The potential benefits of the Appendix A6 provisions tend to be smaller for I-sections with webs that approach the noncompact web slenderness limit, λ_{rw}. In such cases, the somewhat simpler and more streamlined provisions of Article 6.10.8 are recommended for use. The potential gains in economy from using the Appendix A6 provisions increase with decreasing web slenderness.

The Engineer is strongly encouraged to utilize the provisions of Appendix A6 for design of straight bridges with limited skew and compact or nearly compact webs (e.g., rolled-beam sections).

Simple Span Bridges:

The provisions of Appendix A6 are not applicable at the strength limit state for the routine simple-span bridges covered by this Guide because simple-span bridges are subject to positive flexure only and are composite at the strength limit state. The provisions of Article A6.3.3 may potentially be used for these bridges to optionally compute the nominal lateral-torsional buckling resistance of the unbraced lengths of a noncomposite section with a compact or noncompact web during construction for use in checking Eq. 6.10.3.2.1-2 to include the beneficial effect of the St. Venant torsional constant, J (see the Discussion of Article 6.10.3.2.1 in this Guide).

Multi-span Continuous Rolled Beam Bridges:

Sections in regions of negative flexure in the routine multi-span continuous rolled beam bridges covered by this Guide typically qualify as compact web sections and should be designed at the strength limit state using the provisions of Article A6 instead of the more conservative provisions of Article 6.10.8. The provisions of Article A6.3.3 may potentially be used for these bridges to optionally compute the nominal lateral-torsional buckling resistance of the unbraced lengths of the noncomposite section during construction for use in checking Eq. 6.10.3.2.1-2 to include the beneficial effect of the St. Venant torsional constant, J (see the Discussion of Article 6.10.3.2.1 in this Guide).

Multi-span Continuous Plate Girder Bridges:

Sections in regions of negative flexure in the routine multi-span continuous plate girder bridges covered by this Guide that qualify as compact or noncompact web sections can be designed at the strength limit state using these provisions instead of the more conservative provisions of Article 6.10.8, particularly if the sections qualify as compact web sections. The provisions of Article A6.3.3 may potentially be used for these bridges to optionally compute the nominal lateral-torsional buckling resistance of the unbraced lengths of a noncomposite section with a compact or noncompact web during construction for use in checking Eq. 6.10.3.2.1-2 to include the beneficial effect of the St. Venant torsional constant, J (see the Discussion of Article 6.10.3.2.1 in this Guide).

For further information on the provisions and application of Appendix A6, consult Section 6.5.6.2.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples illustrating strength limit state design flexure checks using Appendix A6, consult the NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. **However, the software currently does not include the capability to design the girders using the provisions of Appendix A6.**

A6.1.1 Sections with Discretely Braced Compression Flanges

Determination of applicability, *Simple Span Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

Simple Span Bridges:

This Article is not applicable to the simple span routine steel I-girder bridges covered by this Guide at the strength limit state because simple span bridges are subject to positive flexure only and are composite at the strength limit state (i.e., the compression flange is continuously braced by the deck). The design of the noncomposite section with a discretely braced top (compression) flange during construction in these bridges is separately covered in Article 6.10.3.2.1 (see the Discussion of Article 6.10.3.2.1 in this Guide).

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

This Article provides the relationship that must be satisfied at the strength limit state for compact or noncompact web sections with discretely braced bottom (compression) flanges designed according to the optional provisions of Appendix A6 in regions of negative flexure in multi-span continuous steel I-girder bridges. Therefore, this Article is conditionally applicable at the strength limit state to the design of the section in regions of negative flexure in the routine steel multi-span continuous I-girder bridges covered by this Guide.

For composite sections in regions of negative flexure, lateral bending does not need to be considered in the top (tension) flange at the strength limit state because the flange is continuously supported by the concrete deck. However, since the bottom (compression) flange is discretely braced, lateral bending must be considered in flexural resistance computations for the section (using the "moment form" of the one-third rule flexural resistance equation). For the routine steel I-girder bridges covered by this Guide, the only source of flange lateral bending stress to be considered at the strength limit state is due to wind loading when checking the Strength load combinations that include wind load effects. Amplification of f_ℓ in the discretely braced compression flange will likely be required (see the Discussion of Article 6.10.1.6 in this Guide). Note that f_ℓ cannot exceed $0.6F_{yf}$ after amplification. M_u and f_ℓ are always taken as positive in sign in Eq. A6.1.1-1. However, when summing dead and live load moments to obtain the total factored major-axis moment, M_u, and total factored lateral bending stresses, f_ℓ, to apply in the equations, the signs of the individual stresses or moments must be considered. If there is no flange lateral bending considered, the term $f_\ell S_{xc}$ drops out of the equation. For a section with a discretely braced compression flange, the one-third rule equation must be checked separately for both flange local buckling and lateral-torsional buckling. When lateral-torsional buckling controls, the moment, M_u, used in checking Eq. A6.1.1-1 is to be determined as the value of the major-axis bending moment at the cross-section where $M_u/R_{pc}M_{yc}$ is maximum in the unbraced length under consideration, including the end cross-sections. R_{pc} is the web plastification factor for the compression flange at the cross-section under consideration determined as specified in Article A6.2.1 or A6.2.2, as applicable (see the Discussion of Articles A6.2.1 and A6.2.2 in this Guide). M_{yc} is the yield moment with respect to the compression flange at the cross-section under consideration determined as specified in Article D6.2 (see the Discussion of Article D6.2 in this Guide). When flange local buckling controls, M_u and f_ℓ used to check Eq. A6.1.1.-1 may be determined as the corresponding values at the section under consideration (see the Discussion of Article 6.10.1.6 in this Guide).

The multiplication of f_ℓ by S_{xc} in Eq. A6.1.1-1 stems from the derivation of this equation. The equation may be expressed in a stress format by dividing both sides by the corresponding elastic section modulus, in which case, Eq. A6.1.1-1 reduces effectively to Eqs. 6.10.3.2.1-2 and 6.10.8.1.1-1 in the limit that the web approaches its noncompact web slenderness limit, λ_{rw}. The elastic section modulus, S_{xc}, in Eq. A6.1.1-1 is defined as M_{yc}/F_{yc}, where M_{yc} is the yield moment with respect to the compression flange calculated as specified in Article D6.2 (see the Discussion of Article D6.2 in this Guide) and F_{yc} is the specified minimum yield strength of the compression flange, so that for a composite section with a web proportioned exactly at λ_{rw}, the flexural resistance given by Appendix A6 will be approximately the same as the flexural resistance given by Article 6.10.8. Slight differences between the resistance predictions may occur for reasons pointed out in the Commentary for Article A6.1.1.

The design of the noncomposite section with a discretely braced top (compression) flange in regions of positive flexure and a discretely braced bottom (compression) flange in regions of negative flexure during construction in these bridges is covered in Article 6.10.3.2.1 (see the Discussion of Article 6.10.3.2.1 in this Guide).

For further information on the provisions and application of Appendix A6, consult Section 6.5.6.2.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples illustrating strength limit state design flexure checks using Appendix A6, consult the NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. **However, the software currently does not include the capability to design the girders using the provisions of Appendix A6.**

A6.1.2 Sections with Discretely Braced Tension Flanges

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

Simple Span Bridges:

This Article is not applicable to the simple span routine steel I-girder bridges covered by this Guide at the strength limit state as simple span bridges are subject to positive flexure only and are composite at the strength limit state; as implied by the title of Appendix A6 and stated in the Discussion of Appendix A6, the provisions of Appendix A6 do not apply to composite I-sections in positive flexure. The design of the noncomposite section with a discretely braced bottom (tension) flange during construction in these bridges is covered in Article 6.10.3.2.2 (see the Discussion of Article 6.10.3.2.2 in this Guide).

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

This Article provides the relationship that must be satisfied at the strength limit state for compact or noncompact web sections with discretely braced top (tension) flanges designed according to the optional provisions of Appendix A6 in regions of negative flexure in multi-span continuous steel I-girder bridges. The top flange in regions of negative flexure would only be discretely braced if the shear connectors are intentionally omitted in regions of negative flexure, which is dependent on the preferences of the Owner-agency but not recommended, and if the Engineer deems that the top flange is not sufficiently encased by the concrete deck. Therefore, this Article is conditionally applicable at the strength limit state to the design of the section in regions of negative flexure in routine steel multi-span continuous I-girder bridges covered by this Guide.

The nominal flexural resistance, M_{nt}, in Eq. A6.1.2-1 based on tension-flange yielding is determined as specified in Article A6.4 (see the Discussion of Article A6.4 in this Guide). Since the top (tension) flange is discretely braced if the conditions stated above are met, lateral bending must be considered in flexural resistance computations for the tension flange (using the moment form of the one-third rule flexural resistance equation). For the routine steel I-girder bridges covered by this Guide (which are neither curved nor significantly skewed), the only source of flange lateral bending stress to be considered at the strength limit state is wind loading occurring under the Strength load combinations that include wind load effects. f_ℓ cannot exceed $0.6F_{yf}$. Amplification of f_ℓ in the tension flange is not required. f_{bu} and f_ℓ are always taken as positive in sign in Eq. A6.1.2-1. However, when summing dead and live load moments to obtain the total factored major-axis moment, M_u, and total factored lateral bending stresses, f_ℓ, to apply in the equations, the signs of the individual stresses or moments must be considered. If there is no flange lateral bending considered, the term $f_\ell S_{xt}$ drops out of the equation.

The multiplication of f_ℓ by S_{xt} in Eq. A6.1.2-1 stems from the derivation of this equation. The equation may be expressed in a stress format by dividing both sides by the corresponding elastic section modulus, in which case, Eq. A6.1.2-1 reduces effectively to Eqs. 6.10.7.2.1-2 and 6.10.8.1.2-1 in the limit that the web approaches its noncompact web slenderness limit, λ_{rw}. The elastic section modulus, S_{xt}, in Eq. A6.1.2-1 is defined as M_{yt}/F_{yt}, where M_{yt} is the yield moment with respect to the tension flange calculated as specified in Article D6.2 (see the Discussion of Article D6.2 in this Guide) and F_{yt} is the specified minimum yield strength of the tension flange, so that for a composite section with a web proportioned exactly at λ_{rw}, the flexural resistance given by Appendix A6 will be approximately the same as the flexural resistance given by Article 6.10.8. Slight differences between the resistance predictions may occur for reasons pointed out in the Commentary for Article A6.1.1.

Note that when M_{yc} is less than or equal to M_{yt} and f_ℓ is equal to zero, the flexural resistance based on the tension flange does not control and Eq. A6.1.2-1 need not be checked.

The design of the noncomposite section with a discretely braced bottom (tension) flange in regions of positive flexure and a discretely braced top (tension) flange in regions of negative flexure during construction in these bridges is covered in Article 6.10.3.2.2 (see the Discussion of Article 6.10.3.2.2 in this Guide).

For further information on the provisions and application of Appendix A6, consult Section 6.5.6.2.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples illustrating strength limit state design flexure checks using Appendix A6, consult the NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. **However, the software currently does not include the capability to design the girders using the provisions of Appendix A6.**

A6.1.3 Sections with Continuously Braced Compression Flanges

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article apply only to a noncomposite section at the strength limit state in regions of positive flexure (i.e., girders in positive flexure at the strength limit state with no shear connectors) that is designed according to the optional provisions of Appendix A6, in which the Engineer deems that the flange is sufficiently encased by the concrete deck; this condition is not part of the definition of the routine steel I-girder bridges covered by this Guide.

A6.1.4 Sections with Continuously Braced Tension Flanges

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

This Article provides the relationship that must be satisfied at the strength limit state for compact or noncompact web sections with continuously braced top (tension) flanges in regions of negative flexure in multi-span continuous steel I-girder bridges designed according to the optional provisions of Appendix A6. A continuously braced flange is anchored to the concrete deck by shear connectors or encased in concrete.

Since the flange is continuously braced, only yielding of the flange is a concern and flange lateral bending stresses need not be considered.

Simple Span Bridges: This Article is not applicable to the routine simple span I-girder bridges covered by this Guide at the strength limit state as simple span bridges are subject to positive flexure only.

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*: This Article is applicable to the design of multi-span continuous bridges in which shear connectors are provided in regions of negative flexure. If shear connectors are intentionally omitted in regions of negative flexure, which is dependent on the preferences of the Owner-agency but not recommended, and if the Engineer deems that the top flange is not sufficiently encased by the concrete deck, then the provisions of this Article would not be applicable.

For further information on the provisions and application of Appendix A6, consult Section 6.5.6.2.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples illustrating strength limit state design flexure checks using Appendix A6, consult the NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. **However, the software currently does not include the capability to design the girders using the provisions of Appendix A6.**

A6.2 WEB PLASTIFICATION FACTORS

A6.2.1 Compact Web Sections

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article define the limiting slenderness ratio, $\lambda_{pw(D_{cp})}$, for a compact web section corresponding to the web slenderness, $2D_{cp}/t_w$, where D_{cp} is the depth of the web in compression at the plastic moment determined according to the provisions of Article D6.3.2 (see the Discussion of Article D6.3.2 in this Guide). The enables the section to develop the full plastic moment resistance, M_p (see the Discussion of Article D6.1 in this Guide), provided that other flange slenderness and lateral torsional bracing requirements are satisfied.

The upper limit of $\lambda_{rw}(D_{cp}/D_c)$ in Eq. A6.2.1-2 is to protect against extreme cases where D_c/D is significantly less than 0.5 (see the Discussion of Article 6.10.6.2.3 in this Guide for further information on the limiting slenderness ratio for a noncompact web, λ_{rw}). In such cases, D_{cp}/D is typically smaller than D_c/D. As such, in certain situations, the web slenderness associated with the elastic cross-section, $2D_c/t_w$, may be larger than λ_{rw}, while the slenderness associated with the plastic cross-section, $2D_{cp}/t_w$, may be smaller than $\lambda_{pw(D_{cp})}$. In other words, the elastic web would be classified as slender at the same time the plastic web would be classified as compact. To guard against such situations and the possibility of theoretical bend-buckling of the web prior to reaching M_p, the upper limit of $\lambda_{rw}(D_{cp}/D_c)$ is placed on $\lambda_{pw(D_{cp})}$.

The web slenderness limit given by Eq. A6.2.1-2 accounts for the higher demands on the web in noncomposite singly symmetric I-sections and in composite I-sections in negative flexure with larger shape factors, M_p/M_y. For sections with a specified minimum yield strength of the compression flange, F_{yc}, equal to 50 ksi, the limiting web slenderness based on D_{cp} is 91 for a shape factor of 1.12 and 64 for a shape factor of 1.30. These limits would generally be satisfied by rolled shapes or plate girders with proportions similar to those of a rolled shape, which would typically be used in a shorter-span bridge (i.e., spans of about 120 ft or less).

This Article also defines the web plastification factors, R_{pc} and R_{pt}, for the compression and tension flange, respectively, of a compact web section (Eqs. A6.2.1-6 and A6.2.1-7). The web plastification factors are essentially effective shape factors that define a smooth linear transition in the maximum flexural resistance between M_p and M_y. For a compact web section, the web plastification factors are equivalent to the cross-section shape factors. Thus, whenever R_{pc} and R_{pt} are used in the appropriate flexural resistance equations, the maximum flexural resistance of a compact web section, M_{max}, will equal the plastic moment, M_p. By using R_{pc} and R_{pt} in the flexural resistance equations, separate flexural resistance equations are not required for compact and noncompact web sections.

For further information on the provisions and application of Appendix A6, consult Section 6.5.6.2.3 of the [Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures]. For design examples illustrating strength limit state design flexure checks using Appendix A6, consult the NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. **However, the software currently does not include the capability to design the girders using the provisions of Appendix A6.**

A6.2.2 Noncompact Web Sections

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article define the limiting slenderness ratio, λ_{rw}, for a noncompact web section corresponding to the web slenderness, $2D_c/t_w$, where D_c is the elastic depth of the web in compression determined according to the provisions of Article D6.3.1 (see the Discussion of Article D6.3.1 in this Guide). Sections exceeding the web slenderness limit for a compact web section given by Eq. A6.2.1-1 (see the Discussion of Article A6.2.1 in this Guide), but with a web slenderness less than or equal to λ_{rw}, are termed noncompact web sections, which have a nominal flexural resistance at the strength limit state that linearly transitions from the plastic moment, M_p, to the moment at first yield, M_y (see the Discussions of Articles D6.1 and D6.2 in this Guide) as a function of the web slenderness. For sections with a specified minimum yield strength of the compression flange, F_{yc}, equal to 50 ksi, the upper and lower limits of λ_{rw} are 111 and 137 (see the Discussion of Article 6.10.6.2.3 in this Guide for further information on the limiting slenderness ratio for a noncompact web, λ_{rw}). Plate-girder sections in a medium-span bridge (e.g., spans in the 120-ft to 150-ft range) may potentially qualify as noncompact web sections. Some larger rolled-beam sections may also potentially qualify as noncompact web sections.

This Article also defines the web plastification factors, R_{pc} and R_{pt}, for the compression and tension flange, respectively, of a noncompact web section (Eqs. A6.2.2-6 and A6.2.2-7). Eqs. A6.2.2-6 and A6.2.2-7 define the linear transition in the maximum potential flexural resistance, M_{max}, of a noncompact web section from M_p to M_y as a function of the web slenderness. As $2D_c/t_w$ approaches the noncompact web section limit, λ_{rw}, the web plastification factors approach values equal to the hybrid factor, R_h (see the Discussion of Article 6.10.1.10.1 in this Guide), and therefore, M_{max} within the appropriate flexural resistance equations approaches a limiting value of R_hM_{yc} or R_hM_{yt}, as applicable. As $2D_{cp}/t_w$ approaches the compact web section limit, $\lambda_{pw(Dcp)}$ (see the Discussion of Article A6.2.1 in this Guide), the web plastification factors approach the cross-section shape factors (Eqs. A6.2.1-6 and A6.2.1-7), and therefore, M_{max} within the appropriate flexural resistance equations approaches a limiting value of M_p. By using R_{pc} and R_{pt} in the flexural resistance equations, separate flexural resistance equations are not required for compact and noncompact web sections.

Upper limits of M_p/M_{yc} and M_p/M_{yt} are placed on R_{pc} and R_{pt}, respectively, in Eqs. A6.2.2-6 and A6.2.2-7. These upper limits will limit the larger of the base resistances, $R_{pc}M_{yc}$ or $R_{pt}M_{yt}$, to M_p for the rare case of an extremely singly symmetric section in which M_{yc} or M_{yt} is greater than M_p. The flange-proportioning limit given by Eq. 6.10.2.2-4 (see the Discussion of Article 6.10.2.2 in this Guide) will generally tend to prevent the use of such sections.

Eq. A6.2.2-8 converts the compact web section slenderness ratio, $\lambda_{pw(D_{cp})}$, defined in terms of D_{cp} to a value that can be used consistently in Eqs. A6.2.2-6 and A6.2.2-7with the web slenderness, λ_w, which is expressed in terms of D_c.

For further information on the provisions and application of Appendix A6, consult Section 6.5.6.2.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples illustrating strength limit state design flexure checks using Appendix A6, consult the NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. **However, the software currently does not include the capability to design the girders using the provisions of Appendix A6.**

A6.3 FLEXURAL RESISTANCE BASED ON THE COMPRESSION FLANGE

A6.3.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*:

Discussion:

This Article directs the Engineer to the Articles within Appendix A6 containing the equations necessary to compute the nominal flexural resistance, M_{nc}, of a section with a discretely braced compression flange based on flange local buckling (see the Discussion of Article A6.3.2 in this Guide) or lateral-torsional buckling (see the Discussion of Article A6.3.3 in this Guide) for use in Eq. A6.1.1-1 at the strength limit state (see the Discussion of Article A6.1.1 in this Guide), or potentially in Eq. 6.10.3.2.1-2 for the noncomposite section during construction (see the Discussion of Article 6.10.3.2.1 in this Guide). The equations must be satisfied for both flange local buckling and lateral-torsional buckling.

The Commentary for Article 6.10.8.2.1 includes presentation of the "basic form of all I-section compression-flange flexural resistance equations." **Designers are strongly encouraged to familiarize themselves with the concepts presented in this Commentary and the associated graph in Figure C6.10.8.2.1-1. Possessing a clear understanding of these fundamental**

concepts is invaluable for understanding the associated provisions of the AASHTO LRFD BDS.

Simple Span Bridges:

This Article is applicable for the simple span steel I-girder bridges covered by this Guide only if the noncomposite section qualifies as a compact web or noncompact web section and the provisions of Article A6.3.3 are used to compute M_{nc} for lateral-torsional buckling during construction for use in checking Eq. 6.10.3.2.1-2 to account for the beneficial effect of the St. Venant torsional constant, J (see the Discussion of Article 6.10.3.2.1 in this Guide).

This Article is not applicable to these bridges at the strength limit state as simple spans are subject to positive flexure only and the top (compression) flange is continuously braced by the concrete deck.

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

For the routine multi-span continuous rolled beam and plate girder bridges covered by this Guide, this Article is applicable for determining M_{nc} for sections designed using the provisions of Appendix A6 and with a discretely braced bottom (compression) flange in regions of negative flexure at the strength limit state for use in Eq. A6.1.1-1 (see the Discussion of Article A6.1.1 in this Guide).

The Article is also applicable if the noncomposite section in regions of positive and negative flexure qualifies as a compact web or noncompact web section and the provisions of Article A6.3.3 are used to compute M_{nc} for lateral-torsional buckling during construction for use in checking Eq. 6.10.3.2.1-2 to account for the beneficial effect of the St. Venant torsional constant, J (see the Discussion of Article 6.10.3.2.1 in this Guide).

For further information on the provisions and application of Appendix A6, consult Section 6.5.6.2.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples illustrating strength limit state design flexure checks using Appendix A6, consult the NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. **However, the software currently does not include the capability to design the girders using the provisions of Appendix A6.**

A6.3.2 Local Buckling Resistance

Determination of applicability, *Simple Span Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article are used to compute the nominal flexural resistance, M_{nc}, of a section with a compact or noncompact web and a discretely braced compression flange based on flange local buckling (FLB) for use in Eq. A6.1.1-1 at the strength limit state (see the Discussion of Article A6.1.1 in this Guide).

FLB is a limit state of buckling of a compression flange within a cross-section and is a function of the compression-flange slenderness, $\lambda_f = b_{fc}/2t_{fc}$. For determining the FLB resistance (refer to Figure C6.10.8.2.1-1), λ_{pf} locates Anchor Point 1 (Eq. A6.3.2-4) that separates sections with compact flanges from sections with noncompact flanges (see Table C6.10.8.2.2-1). A member with a compression-flange slenderness at or below the compact flange limit, λ_{pf}, is able to achieve the so-called "plateau strength" or maximum potential FLB resistance (M_{max} in Figure C6.10.8.2.1-1) of R_pM_{yc} (Eq. A6.3.2-1), which is independent of the compression-flange slenderness. λ_{rf} locates Anchor Point 2 (Eq. A6.3.2-5) that separates sections with noncompact flanges from sections with slender flanges and is the point where the inelastic and elastic FLB resistances are the same (with the resistance at this point assumed to be $R_bF_{yr}S_{xc}$). F_{yr} for checking FLB is the compression-flange stress at the onset of nominal yielding within the cross-section, including residual stress effects, but not including compression flange lateral bending, taken as the smaller of $0.7F_{yc}$, $R_hF_{yt}S_{xt}/S_{xc}$ and F_{yw}, but not less than $0.5F_{yc}$. The section moduli, S_{xc} and S_{xt}, are calculated as specified in this Article. The inelastic FLB resistance of a noncompact flange is treated as a linear function of the compression-flange slenderness (Eq. A6.3.2-2). An elastic FLB equation for slender flanges is not provided in the specifications because for most practical bridge-girder sections, including sections used in the routine I-girder bridges covered by this Guide (i.e., with $F_{yc} \leq 90$ ksi), elastic FLB will not control as λ_f is limited to a practical maximum value of 12.0 (see the Discussion of Article 6.10.2.2 in this Guide). The FLB resistance for moment gradient cases is treated the same as that for the case of uniform major-axis bending; i.e., the relatively minor influence of moment-gradient effects on the FLB resistance is neglected.

Eq. A6.3.2-5 for the anchor point, λ_{rf}, includes a flange local buckling coefficient, k_c. Eq. A6.3.2-6 provides an expression for k_c for built-up sections. This expression accounts for the fact that thinner webs in built-up sections tend to offer less rotational restraint to prevent flange local buckling. The calculated value of k_c from Eq. A6.3.2-6 must fall between the range of 0.35 and 0.76. The upper-bound value of 0.76 corresponds to the traditional value that has been assumed for rolled shapes in the AISC *Specification for Structural Steel Buildings*. The lower bound k_c value of 0.35 is conservatively assumed for all sections in Eq. 6.10.8.2.2-5 for λ_{rf}, which is assumed to apply only to slender web sections.

For design checks where the flexural resistance is based on FLB, M_u and f_ℓ in in Eq. A6.1.1-1 are determined as the moment and stress at the section under consideration (see the Discussion of Article 6.10.1.6 in this Guide).

Simple Span Bridges:

The provisions of this Article are not applicable to the routine simple span bridges covered by this Guide at the strength limit state because simple spans are subject to positive flexure and the top (compression) flange is continuously braced by the concrete deck. These provisions are also not applicable to these bridges during construction because the FLB resistance of the noncomposite section during construction is to be checked using Eq. 6.10.3.2.1-2 in conjunction with the equations of Article 6.10.8.2.2 (see the Discussions of Articles 6.10.3.2.1 and 6.10.8.2.2 in this Guide).

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges:*

The provisions of this Article are conditionally applicable for the routine steel multi-span continuous I-girder bridges covered by this Guide to calculate the nominal FLB resistance of sections with a compact or noncompact web in regions of negative flexure at the strength limit state with a discretely braced bottom (compression) flange for use in Eq. A6.1.1-1 (if the section is designed using the provisions of Appendix A6).

For further information on the provisions and application of Appendix A6, consult Section 6.5.6.2.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples illustrating strength limit state design flexure checks using Appendix A6, consult the NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. **However, the software currently does not include the capability to design the girders using the provisions of Appendix A6.**

A6.3.3 LTB Resistance

A6.3.3.1 General

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article are used to compute the nominal flexural resistance, M_{nc}, of a section with a compact or noncompact web and a discretely braced compression flange based on lateral-torsional buckling (LTB) for use in Eq. A6.1.1-1 or 6.10.3.2.1-2, as applicable (see the Discussion of Articles A6.1.1 and 6.10.3.2.1 in this Guide). The provisions address the calculation of the nominal LTB resistance for general composite or noncomposite unbraced lengths with single- or reverse curvature bending, double- or single-symmetry of the steel cross-section, and prismatic or nonprismatic geometry, including potential steps in F_{yc} along the length. The provisions are not applicable for I-section members subjected to single-curvature bending in which the flange in flexural compression is continuously braced within the entire unbraced length under consideration. For these types of unbraced lengths, the continuously braced flange is to be checked by the provisions of Article 6.10.7.1, 6.10.7.2, or 6.10.8.1.3, as applicable (see the Discussion of Articles 6.10.7.1, 6.10.7.2, and 6.10.8.1.3 in this Guide). The last paragraph of the Commentary of this Article addresses unbraced lengths in single-curvature bending where continuous bracing of a flange subjected to compression ends. For cases where a shortened unbraced length is used, points A, B, and C used in the computation of the term C_b in Article 6.10.8.2.3b (see the Discussion of Article 6.10.8.2.3b in this Guide) should be taken as the quarter points of the shortened unbraced length. Such a situation would be very uncommon in the routine steel I-girder bridges covered by this Guide. Refer to the last paragraph of this Article for special considerations for unbraced lengths subject to reverse-curvature bending. This Article is conditionally applicable for routine steel multi-span continuous I-girder bridges, and only partially applicable for routine simple span I-girder bridges covered by this Guide, as described further in the Discussion of Article A6.3.1 in this Guide. If a member has a slender web at any location along the unbraced length under consideration, it is to be considered as a slender-web member along the entire unbraced length and the provisions of Article 6.10.8.2.3 are to apply (see the Discussion of Article 6.10.8.2.3 in this Guide). See the Commentary for Article 6.10.6.2.3 and the Discussion of Article 6.10.6.2.3 in this Guide for further discussion on the definition and categorization of compact web, noncompact web, and slender web sections.

LTB is a limit state of buckling of an unbraced length involving lateral deflection and twist and is a function of the unbraced length, L_b. For determining the LTB resistance (refer to Figure C6.10.8.2.1-1), L_p locates Anchor Point 1 that separates compact unbraced lengths from noncompact unbraced lengths (Eq. A6.3.3.1-4). A member braced at or below the compact unbraced length limit is able to achieve the so-called “plateau strength” or maximum potential LTB resistance (M_{max} in Figure C6.10.8.2.1-1) of $R_{pc}M_{yc}$ under uniform bending, which is independent of the unbraced length (Eq. A6.3.3.1-1). Note that in many cases, it will not be economical to brace the girder at a distance equal to L_p or below to reach M_{max}, particularly under uniform bending conditions for which C_b is equal to 1.0. L_r locates Anchor Point 2 that separates sections with noncompact unbraced lengths from sections with slender unbraced lengths (Eq. A6.3.3.1-5) and is the point where the inelastic and elastic LTB resistances are the same (with the resistance at this point assumed to be $R_bF_{yr}S_{xc}$). F_{yr} for checking LTB is the compression-flange stress at the onset of nominal yielding, including residual stress and geometric imperfection effects but not including compression-flange lateral bending, and is to be taken as $0.5F_{yc}$ for welded-section members. The preceding value of F_{yr} provides a more uniform level of reliability consistent with the target levels in the AISC and AASHTO LRFD Specifications compared with the variable value of F_{yr}, often equal to $0.7F_{yc}$, in editions of the AASHTO LRFD BDS prior to the 10th Edition. The value of $0.7F_{yc}$ is retained for rolled-section members. The inelastic LTB resistance of a noncompact unbraced length is treated as a linear function of the unbraced length (Eq. A6.3.3.1-2). Unbraced lengths greater than L_r are termed slender unbraced lengths, and their resistance is controlled by elastic LTB (Eq. A6.3.3.1-3). LTB in the elastic range is of primary importance for relatively slender girders braced at longer than normal intervals, which most typically occurs during a temporary construction condition. Note that the parameter, S_{xc}, to be used in the computation of L_r in Eq. A6.3.3.1-5 is to be taken as the elastic section modulus about the major axis of the section to the compression flange; otherwise, it is to be taken as M_{yc}/F_{yc} in the calculation of the nominal LTB resistance in Eqs. A6.3.3.1-2 and A6.3.3.1-3, where M_{yc} is the yield moment with respect to the compression flange determined as specified in Article D6.2 (see the Discussion of Article D6.2 in this Guide).

Article A6.3.3.2 applies for determining the LTB parameters C_b, F_e, and r_t to substitute in these nominal LTB resistance equations for prismatic unbraced lengths. Article A6.3.3.3 applies for determining these parameters for nonprismatic unbraced lengths (see the Discussions of Articles A6.3.3.2 and A6.3.3.3 in this Guide).

In lieu of a more refined analysis, the nominal flexural resistance based on LTB is to be calculated at the so-called "governing cross-section", which is the cross-section where $M_u/R_{pc}M_{yc}$ is maximum in the unbraced length under consideration, including the end cross-sections. M_u is the factored major-axis bending moment at the cross-section under consideration. R_{pc} is the web plastification factor for the compression flange at the cross-section under consideration determined as specified in Article A6.2.1 or A6.2.2, as applicable (see the Discussion of Articles A6.2.1 and A6.2.2 in this Guide). M_{yc} is the yield moment with respect to the compression flange at the cross-section under consideration determined as specified in Article D6.2 (see the Discussion of Article D6.2 in this Guide). The Commentary for this Article further discusses the various aspects of the determination of the governing cross-section.

For design checks where the flexural resistance is based on LTB, the moment M_u in Eq. A6.1.1-1 or Eq. 6.10.3.2.1-2, as applicable, is to be determined as the value of the major-axis bending moment at the cross-section where $M_u/R_{pc}M_{yc}$ is maximum in the unbraced length under consideration, including the end cross sections, and f_ℓ is to be taken as the largest value of the stress due to lateral bending throughout the unbraced length in the flange under consideration. Combined vertical and flange lateral bending is addressed in the Specifications by effectively handling the flanges as equivalent beam-columns. The use of maximum values within the unbraced length, when the resistance is governed by member stability, i.e., LTB, is consistent with established practice in the proper application of beam-column interaction equations (see the Discussion of Article 6.10.1.6 in this Guide and the Commentary for Article 6.10.8.2.3a).

The effective length factor, K, for LTB is assumed equal to 1.0 in the equations of this Article. The Commentary of this Article discusses the possibility of obtaining a more refined estimate of the LTB resistance accounting for end restraint from adjacent unbraced lengths and/or connection details through the calculation of an effective unbraced length, KL_b, which may be substituted for the length, L_b, in the LTB equations of this article. References to a simple hand method for calculation of elastic LTB effective length factors are provided (see Appendix A of the NSBA's Steel Bridge Design Handbook – Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge for an example calculation using this method). Alternatively, a more refined estimate of the LTB resistance may be obtained from a direct buckling analysis (see the Commentary for Article D6.6.4 and the AASHTO Nonprismatic Girder Design Guide for guidance pertaining to this type of analysis). Typically, neither of these approaches are employed or necessary in the design of the routine steel I-girder bridges covered by this Guide. However, the application of these methods may potentially be advantageous in load rating.

The 10th Edition of the AASHTO LRFD BDS also introduced provisions and commentary related to the use of half-round I-girder bearing stiffeners at supports. The Commentary of this Article suggests that the use of half-round I-girder bearing stiffeners can provide increased torsional warping restraint, resulting in a beneficial increase in lateral-torsional buckling resistance locally in the unbraced length containing this type of stiffener, and provides simplified guidance for quantifying this beneficial effect. The use of half-round I-girder bearing stiffeners is typically reserved for cases of fairly severe skew but may be considered on some moderately skewed steel I-girders bridges when appropriate. See the Discussion of Article 6.10.11.2 and the associated sub-articles for more information on half-round I-girder bearing stiffeners.For further information on the provisions and application of Appendix A6, consult Section 6.5.6.2.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples illustrating strength limit state design flexure checks using Appendix A6, consult the NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. **However, the software currently does not include the capability to design the girders using the provisions of Appendix A6.**

A6.3.3.2 LTB Parameters for Prismatic Unbraced Lengths

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

Article A6.3.3.2 provides equations for determining the LTB parameters C_b, F_e, and r_t to substitute in the LTB nominal resistance equations of Article A6.3.3.1 for prismatic unbraced lengths (see the Discussion of Article A6.3.3.1 in this Guide) of compact-web and noncompact-web section members. A prismatic unbraced length is defined as an unbraced length between cross-frames or diaphragms in which the member cross-section and yield strength does not vary along the length.

The provisions of this Article are conditionally applicable for routine steel multi-span continuous I-girder bridges, and only partially applicable for routine simple span I-girder bridges covered by this Guide, as described further in the Discussion of Article A6.3.1 in this Guide. If a member has a slender web at any location along the unbraced length under consideration, it is to be considered as a slender-web member along the entire unbraced length and the provisions of Article 6.10.8.2.3

are to apply (see the Discussion of Article 6.10.8.2.3 in this Guide). See the Commentary for Article 6.10.6.2.3 and the Discussion of Article 6.10.6.2.3 in this Guide for further discussion on the definition and categorization of compact web, noncompact web, and slender web sections.

The solid curve in Figure C6.10.8.2.1-1 is for LTB under uniform bending and is represented by the equations given in Article A6.3.3.1 (see the Discussion of Article A6.3.3.1 in this Guide). The dashed curve in Figure C6.10.8.2.1-1 shows the solid curve scaled by the moment-gradient modifier, C_b, under moment-gradient conditions, which can result in the plateau strength (M_{max}) for lateral-torsional buckling to be reached at significantly larger unbraced lengths under moment-gradient conditions when the effects of the moment gradient are included in determining the limits on the unbraced length, L_b. Refer to Article D6.4.2 for the appropriate equations to use under these conditions (i.e., the equations representing the dashed curve when C_b is calculated and is greater than 1.0), which is strongly encouraged for prismatic unbraced lengths (see the Discussion of Article D6.4.2 in this Guide).

C_b accounts for the effect of a variation in moment along the unbraced length. For prismatic unbraced lengths, C_b is to be calculated from Eq. 6.10.8.2.3b-1, which requires the input of the absolute value of the factored major-axis bending moment at one-quarter, one-half, and three-quarters of the unbraced length under consideration (points A, B, and C), respectively, calculated from the moment envelope values that produce the largest flexural compression in the flange under consideration at these points, or the smallest flexural tension in this flange if the flange is never in compression at the point. For single-curvature bending, there is no upper limit on the value of C_b computed from this equation. Since concurrent moments are normally not tracked in the analysis, it is convenient and considered acceptable to utilize the factored worst-case moments from the live load moment envelopes in conjunction with other factored moment diagrams for calculation of C_b. M_{max} in Eq. 6.10.8.2.3b-1 is the absolute value of the factored maximum major-axis moment in the unbraced length, calculated from the moment envelope value that produces the largest flexural compression in the flange under consideration. For points A, B, or C in unbraced lengths of noncomposite or composite section members where the flange under consideration is subjected to compression and is continuously braced anywhere within either quarter portion of the unbraced length adjacent to the point under consideration, the moment corresponding to that point, A, B, or C, is to be taken equal to zero in Eq. 6.10.8.2.3b-1.

The bulleted items at the beginning of the Commentary for Article C6.10.8.2.3b indicate the conditions for which Eq. 6.10.8.2.3b-1 is considered applicable for the computation of C_b. For prismatic noncomposite unbraced lengths of singly symmetric members subject to reverse curvature bending, Eq. 6.10.8.2.3b-1 should be multiplied by the factor, R_m, calculated from Eq. C6.10.8.2.3b-1 or C6.10.8.2.3b-2, as applicable. The resulting value of C_b from this calculation is not to exceed 3.0. Alternatively, the Commentary points to the provisions of Article 6.10.8.2.3c to calculate the LTB parameters for a more refined and potentially accurate solution for prismatic noncomposite unbraced lengths of singly symmetric members subject to reverse curvature bending and for prismatic composite unbraced lengths subject to reverse curvature bending, with the elastic LTB load ratio, γ_e, calculated using Method A specified in Article D6.6.2 (see the discussion of Articles 6.10.8.2.3c and D6.6.2 in this Guide).

C_b from Eq. 6.10.8.2.3b-1 has a base value of 1.0 when the moment and the corresponding flange compressive major-axis bending stress are constant over the unbraced length. C_b may be conservatively taken equal to 1.0 for all cases, with the exception of unusual circumstances involving no bracing within the span and significant top flange loading, and cantilever beams with flexible backspans and/or significant top-flange loading. The 7th and 8th paragraphs of the Commentary to Article C6.10.8.2.3b provide recommendations for what to do in such cases.

Eq. A6.3.3.2-1 for the elastic LTB stress, F_e, at the governing cross-section (see the discussion of Article A6.3.3.1 in this Guide) is the exact beam-theory solution for the elastic LTB resistance of a doubly symmetric I-section under uniform bending (assuming load-height effects are not considered). The radius of gyration for LTB at the governing cross-section, r_t, used in the calculation of F_e is determined from Eq. 6.10.8.2.3b-3. Alternatively, Eq. C6.10.8.2.3b-4 in the Commentary for Article 6.10.8.2.3b is permitted for use for a more exact calculation of r_t for software calculations or if the Engineer requires a more precise calculation of F_e. Eq. 6.10.8.2.3b-3 is a simplification of Eq. C6.10.8.2.3b-4 obtained by taking $D = h = d$. Further details on the accuracy of this approximation in the computation of F_e are discussed in the Commentary for this Article. Unlike Eqs. 6.10.8.2.3b-2 and 6.10.8.2.3a-5 for F_e and L_r, respectively, which assume slender-web behavior, the equations for F_e and L_r in Appendix A6 include the St. Venant torsional constant J, which is appropriate for stockier compact web and noncompact web sections that are generally not subject to significant web distortion. For welded-sections, J may be computed from Eq. A6.3.3.1-6, which provides an accurate approximation of the constant neglecting the effect of the web-to-flange fillets (for rolled beams, refer to the tabulated AISC Manual values of J instead, which include the effect of the web-to-flange fillets). For a compression or tension flange with a ratio, b_f/t_f, greater than 15, the term in parentheses given in Eq. A6.3.3.1-6 for that flange may be taken equal to one. The Commentary to this Article spells out the unusual conditions for which J from Eq. A6.3.3.1-6 may be factored by 0.8 to account for the tendency of Eq. A6.3.3.2-1 to overestimate the LTB resistance in such cases, which should typically not be necessary for the routine steel I-girder bridges covered by this Guide. Note that when J is set to zero, Eq. A6.3.3.2-1 for F_e reduces to Eq. 6.10.8.2.3b-2 and Eq. A6.3.3.1-5 for L_r reduces to Eq. 6.10.8.2.3a-5. S_{xc} in Eq. A6.3.3.2-1 is to be taken as the elastic section modulus about the major axis of the section to the compression flange.

For highly singly symmetric I-sections with a smaller compression flange or for composite I sections in negative flexure, Eq. A6.3.3.2-1 is somewhat conservative compared to rigorous beam-theory based solutions since the equation does not account for the restraint against lateral buckling of the compression flange provided by the larger tension flange, or the deck at the level of the top flange in a composite I-girder. For compact-web and noncompact-web I-section members, the torsional restraint provided by the bridge deck may provide a significant enhancement to the elastic LTB resistance, or more properly, the elastic lateral distortional buckling resistance of the members. Assuming that the member is adequately braced at the cross-frame locations, this influence may be considered for constant web depth members. The Commentary for this Article provides equations to estimate the elastic lateral distortional buckling stress of the bottom compression flange of a composite unbraced length, which may then be employed in place of F_e in Eq. A6.3.3.1-3 to recognize the benefits of the torsional restraint from the bridge deck and the distortional stiffness of the I-section web.

Simple Span Bridges:

For simple span bridges, the moment-gradient modifier, C_b, may conservatively be taken equal to 1.0 when checking LTB of the critical noncomposite unbraced length in regions of positive flexure during construction.

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

For multi-span continuous bridges, it is strongly recommended that as a minimum the moment-gradient modifier, C_b, be calculated from Eq. 6.10.8.2.3b-1 when checking LTB of the first unbraced length on either side of the interior piers, specifically for prismatic unbraced lengths or for nonprismatic unbraced lengths satisfying the 20 percent rule described below in the Discussion of Article 6.10.8.2.3c in this Guide. The unbraced lengths on either side of the pier should be checked to determine which side will yield the lower value of C_b. The provisions of Article D6.4.2 should then be employed to determine the shift in the anchor point, L_p, and the corresponding nominal LTB resistance for these unbraced lengths (see the Discussion of Article D6.4.2 in this Guide).

C_b may conservatively be taken equal to 1.0 when checking LTB of the critical noncomposite unbraced length in regions of positive flexure in multi-span continuous bridges during construction.

For further information on the provisions and application of Appendix A6, consult Section 6.5.6.2.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples illustrating strength limit state design flexure checks using Appendix A6, consult the NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. **However, the software currently does not include the capability to design the girders using the provisions of Appendix A6.**

A6.3.3.3 LTB Parameters for Nonprismatic Unbraced Lengths

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

Article A6.3.3.3 provides equations for determining the LTB parameters C_b, F_e, and r_t to substitute in the LTB nominal resistance equations of Article A6.3.3.1 for nonprismatic unbraced lengths (see the Discussion of Article A6.3.3.1 in this Guide) of compact-web and noncompact-web members. A nonprismatic unbraced length is defined as an unbraced length between cross-frames or diaphragms in which the member cross-section and/or yield strength varies along the length. The provisions of Article D6.4.2 (see the Discussion of Articles A6.3.3.2 and D6.4.2 in this Guide) are not to be employed for nonprismatic unbraced lengths.

The provisions of this Article are conditionally applicable for routine steel multi-span continuous I-girder bridges, and only partially applicable for routine simple span I-girder bridges covered by this Guide, as described further in the Discussion of Article A6.3.1 in this Guide. If a member has a slender web at any location along the unbraced length under consideration, it is to be considered as a slender-web member along the entire unbraced length and the provisions of Article 6.10.8.2.3 are to apply (see the Discussion of Article 6.10.8.2.3 in this Guide). See the Commentary for Article 6.10.6.2.3 and the Discussion of Article 6.10.6.2.3 in this Guide for further discussion on the definition and categorization of compact web, noncompact web, and slender web sections.

The Article presents provisions to directly calculate the LTB resistance of nonprismatic unbraced lengths. However, the application of those provisions is fairly involved and complicated, so the Article also allows for treatment of a nonprismatic unbraced length as prismatic if certain geometric conditions and proportional limits are met. For new design of the routine steel I-girder bridges covered by this Guide, it is appropriate, reasonable, and recommended to configure the design of the girders to meet the requirements that allow for simplified treatment of a non-prismatic unbraced length as prismatic. The more complicated and involved procedure for directly calculating the LTB resistance of nonprismatic unbraced lengths should be reserved for load rating of existing structures or the design of non-routine bridges.

In this Discussion, the provisions that allow for treatment of a nonprismatic unbraced length as prismatic are presented first. For unbraced lengths of constant web depth members containing a single cross-section transition to a smaller section at a distance less than or equal to 20 percent of the unbraced length from the brace point with the smaller magnitude of the moment, and where the larger magnitude of the moment occurs within the section with the largest resistance, and where the ratios of the lateral moments of inertia, I_{yt1}/I_{yt2} and I_{yb1}/I_{yb2}, of the adjacent top and bottom flanges, respectively, of each section at the transition are greater than or equal to 0.5 (where Flange 1 is the flange located closer to the brace point with the smaller magnitude of moment and Flange 2 is the flange located closer to the larger magnitude of moment), the LTB resistance of the compression flange, F_{nc}, may be determined assuming the unbraced length is prismatic using the parameters calculated as specified in Article A6.3.3.2 (see the Discussion of Article A6.3.3.2 in this Guide); that is, assuming that the transition does not exist and that the flanges of the section closer to the brace point with the larger magnitude of moment are extended to the brace point with the smaller magnitude of moment.

For the case of uniform bending, the reduction in the elastic LTB resistance due to a cross-section transition located within the unbraced length of a constant web depth member is approximately five percent when the transition is placed with 20 percent of the unbraced length from the brace point with the smaller magnitude of moment and the preceding flange moment of inertia requirements are satisfied. The moment gradient modifier, C_b, from Article 6.10.8.2.3b should be calculated and applied in this case (see the Discussion of Article 6.10.8.2.3b in this Guide) and L_b

may also be modified by an effective length factor, if desired (see the Discussion of Article A6.3.3.1 in this Guide). Otherwise, the LTB resistance of the unbraced length is to be determined as specified in this Article.

For a constant web depth member with more than one transition, all transitions located at or closer than 20 percent of the unbraced length from the brace point with the smaller magnitude of moment, and with the ratio of the lateral moments of inertia, I_{yt1}/I_{yt2} and I_{yb1}/I_{yb2}, of the adjacent flanges of each section (i.e., $t_f b_f^3/12$) equal to or larger than 0.5, may be ignored. In such cases, the LTB resistance of the remaining prismatic or nonprismatic unbraced length may then be computed as specified in Article A6.3.3.2 or Article A6.3.3.3, as applicable, based on the remaining sections. In addition, any adjacent section transitions, involving stepping the thickness of the web or the area of the flanges, closer than 25 percent of the unbraced length from each other should be considered to all be a part of the same section transition. The "equivalent" single section transition should be taken as located at the flange transition furthest from the closest brace point. Where a cross-section transition within the unbraced length occurs at a bolted field splice, refer to this Article regarding the calculation of I_{yt1} and I_{yb1}, the location of the transition, and the minimum length and contribution of the flange splice plates.

For nonprismatic unbraced lengths with transitions located further than 20 percent of the unbraced length from the brace point with the smaller magnitude of moment, or not satisfying the previously mentioned moment of inertia proportioning requirements, this Article includes provisions for the direct calculation of the LTB resistance of the nonprismatic unbraced length.

In this Article, an equivalent r_t for LTB at the governing cross-section (see the discussion of Article A6.3.3.1 in this Guide) is computed from Eq. A6.3.3.3-3, which allows for the direct use of Eqs. A6.3.3.1-1 through A6.3.3.1-3 to compute the nominal LTB resistance of the nonprismatic unbraced length. The equivalent r_t is computed from the elastic LTB stress, F_e, determined from Eq. A6.3.3.3-2. Note that when J is set to zero, Eq. A6.3.3.3-3 for r_t reduces to Eq. 6.10.8.2.3c-3. F_e is determined as the product of $f_{bu}= M_u S_{xc}$, calculated at the cross-section where $M_u/R_{pc}M_{yc}$ is maximum in the unbraced length under consideration, including the end cross-sections (i.e., the governing cross-section), and an elastic LTB load ratio, γ_e, which is a constant by which the calculated design moments at the governing cross-section would need to be scaled to reach the theoretical elastic LTB load level (refer to Figure CA6.3.3.3-1 in the Commentary of this Article). M_u is the factored major-axis bending moment at the cross-section under consideration. S_{xc} is to be taken as the elastic section modulus about the major axis of the section to the compression flange. R_{pc} is the web plastification factor for the compression flange at the cross-section under consideration determined as specified in Article A6.2.1 or A6.2.2, as applicable (see the Discussion of Articles A6.2.1 and A6.2.2 in this Guide). M_{yc} is the yield moment with respect to the compression flange at the cross-section under consideration determined as specified in Article D6.2 (see the Discussion of Article D6.2 in this Guide).

The elastic LTB load ratio, γ_e, may be calculated using one of the three alternative methods specified in Article D6.6 (i.e., Method A, Method B, or Method C - see the discussion of Article D6.6 in this Guide). There is no particular favor given to any of the alternative methods to calculate γ_e. The designer is free to evaluate each method and choose which one is easier to use, better suited to the situation at hand, etc. The methods should give reasonably comparable results in most cases.

The more approximate Methods A and B were determined to be viable and are just different approaches to investigate a very complex problem in a reasonable fashion. The methods do not supersede each other. For cases where the elastic LTB resistance is calculated directly from an elastic eigenvalue buckling analysis (i.e., Method C), γ_e is to be taken as the smallest, or controlling, eigenvalue obtained from the buckling solution. Since moment-gradient effects are directly considered in the computation of γ_e by Methods A, B, or C, the moment-gradient modifier, C_b, is to be taken equal to 1.0 (Eq. A6.3.3.3-1) whenever the provisions of this Article A6.3.3.3 are employed to compute the nominal LTB resistance.

For further information and design examples illustrating the application of the LTB provisions for nonprismatic unbraced lengths using Methods A, B, and C, consult the AASHTO Nonprismatic Girder Design Guide. For further information on the provisions and application of Appendix A6, consult Section 6.5.6.2.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples illustrating strength limit state design flexure checks using Appendix A6, consult the NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. **However, the software currently does not include the capability to design the girders using the provisions of Appendix A6.**

A6.4 FLEXURAL RESISTANCE BASED ON TENSION FLANGE YIELDING

Determination of applicability, *Simple Span Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

Simple Span Bridges:

This Article is not applicable to the routine simple span I-girder bridges covered by this Guide at the strength limit state because simple span bridges are subject to positive flexure only. The design of the noncomposite section with a discretely braced bottom (tension) flange during construction in these bridges is covered in Article 6.10.3.2.2 (see the Discussion of Article 6.10.3.2.2 in this Guide).

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

The provisions of this Article are used to compute the nominal flexural resistance, M_{nt}, of a section with a discretely braced top (tension) flange at the strength limit state in regions of negative flexure in multi-span continuous steel I-girder bridges for use in Eq. A6.1.2-1 (see the Discussion of Article A6.1.2 in this Guide). The nominal flexural resistance is based only on nominal yielding because flange local buckling and lateral-torsional buckling are not a consideration for flanges in tension. The top flange in regions of negative flexure would only be discretely braced if the shear connectors are intentionally omitted in regions of negative flexure, which is dependent on the preferences of the Owner-agency but not recommended, and if the Engineer deems that the top flange is not sufficiently encased by the concrete deck. Therefore, this Article is conditionally applicable at the strength limit state to the design of the section in regions of negative flexure in routine steel multi-span continuous I-girder bridges, as described further in the Discussion of Article A6.1.2 in this Guide.

Eq. A6.4-1 represents a linear transition in the flexural resistance between M_{yt} and M_p as a function of the web slenderness, $2D_c/t_w$. As the web slenderness approaches the noncompact web section limit, λ_{rw} (see the Discussion of Article A6.2.2 in this Guide), Eq. A6.4-1 approaches the nominal flexural resistance based on tension flange yielding equal to R_hF_{yt}.

For sections in which $M_{yt} > M_{yc}$, Eq. A6.4-1 does not control and need not be checked.

For further information on the provisions and application of Appendix A6, consult Section 6.5.6.2.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. For design examples illustrating strength limit state design flexure checks using Appendix A6, consult the NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

The NSBA's LRFD Simon line-girder analysis and design software available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. **However, the software currently does not include the capability to design the girders using the provisions of Appendix A6.**

APPENDIX B6 MOMENT REDISTRIBUTION FROM INTERIOR-PIER I-SECTIONS IN STRAIGHT CONTINUOUS-SPAN BRIDGES

B6.1 GENERAL

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this optional Appendix B6 may be used to calculate redistribution moments due to the effects of yielding from the interior-pier sections of straight continuous-span I-section flexural members at the service and/or strength limit states. The provisions of Appendix B6 are not applicable to simple-span bridges since the application of these provisions necessarily implies the formation of a plastic hinge in the superstructure at the strength limit state. The formation of a plastic hinge in a simple-span structure would represent a collapse mechanism. Simple-span bridges also do not have interior piers.

These provisions provide an approximate procedure and a refined method to calculate the redistribution moments, which both utilize elastic moment envelopes and do not require the direct use of inelastic analysis methods. The provisions may only be applied to straight continuous-span I-section members satisfying specified limitations (see the Discussion of Article B6.2 in this Guide). The provisions of Articles B6.3 and B6.4 are used to calculate the redistribution moments using the approximate procedure and the provisions of Article B6.6 are used to calculate the redistribution moments using the refined method (see the Discussion of these Articles in this Guide).

Allowing redistribution of negative moments in multi-span continuous steel I-girder bridges can potentially produce more economical designs, but the associated analysis and design considerations are unfamiliar to most Engineers and most Owner-agencies currently do not permit or encourage the use of moment redistribution methods. As a result, the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges.

See the Discussion of Article 4.6.4.1 in this Guide for a basic explanation of moment redistribution methods and their associated advantages and disadvantages. For further information on the provisions of Appendix B6, consult Section 6.5.6.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures, as well as NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

B6.2 SCOPE

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article define the scope of the straight continuous-span I-section members to which the provisions of Appendix B6 may be applied. Specifically, the provisions of Appendix B6 may be applied only to straight continuous I-section members whose support lines are not skewed more than 10 degrees from normal and along which there are no staggered cross-frames. Cross-sections throughout the unbraced lengths immediately adjacent to interior-pier sections from which moments are redistributed must also satisfy certain specified restrictions defined in subsequent Articles to demonstrate adequate robustness to redistribute the moments. If the approximate procedure is used to calculate the redistribution moments, the unbraced lengths adjacent to interior-pier sections must satisfy these restrictions. If the refined method is used to calculate the redistribution moments, the unbraced lengths adjacent to interior-pier sections need not satisfy these restrictions; however, moments may not be redistributed from interior-pier sections that do not satisfy these restrictions.

Although members in routine steel I-girder bridges covered by this Guide may satisfy these restrictions, most Owner-agencies currently do not permit or encourage the use of moment redistribution methods. As a result, the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges.

See the Discussion of Article 4.6.4.1 in this Guide for a basic explanation of moment redistribution methods and their associated advantages and disadvantages. For further information on the provisions of Appendix B6, consult Section 6.5.6.6 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures, as well as NSBA's Steel Bridge Design Handbook – Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge, and NSBA's Steel Bridge Design Handbook – Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge. The reader is cautioned that these references have not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; they still contain significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

B6.2.1 Web Proportions

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article define three web proportioning limits that must be satisfied within the unbraced lengths immediately adjacent to interior-pier sections from which moments are redistributed to apply the optional moment redistribution provisions of Appendix B6.

These provisions are not applicable since the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges (see the Discussion of Article B6.1in this Guide).

B6.2.2 Compression Flange Proportions

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article define two compression flange proportioning limits that must be satisfied within the unbraced lengths immediately adjacent to interior-pier sections from which moments are redistributed to apply the optional moment redistribution provisions of Appendix B6.

These provisions are not applicable since the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges (see the Discussion of Article B6.1in this Guide).

B6.2.3 Section Transitions

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provision in this Article requires unbraced lengths immediately adjacent to interior-pier sections from which moments are redistributed to be prismatic to apply the optional moment redistribution provisions of Appendix B6.

This provision is not applicable since the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges (see the Discussion of Article B6.1in this Guide).

B6.2.4 Compression Flange Bracing

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provision in this Article defines a compression flange bracing limitation that must be satisfied within the unbraced lengths immediately adjacent to interior-pier sections from which moments are redistributed to apply the optional moment redistribution provisions of Appendix B6.

This provision is not applicable since the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges (see the Discussion of Article B6.1in this Guide).

B6.2.5 Shear

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provision in this Article limits the maximum factored shear within the unbraced lengths immediately adjacent to interior-pier sections from which moments are redistributed to the shear-yield or shear-buckling resistance to apply the optional moment redistribution provisions of Appendix B6.

This provision is not applicable since the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges (see the Discussion of Article B6.1in this Guide).

B6.2.6 Bearing Stiffeners

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provision in this Article requires bearing stiffeners to be placed at interior-pier sections from which moments are redistributed to apply the optional moment redistribution provisions of Appendix B6.

This provision is not applicable since the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges (see the Discussion of Article B6.1in this Guide).

B6.3 SERVICE LIMIT STATE

B6.3.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provision in this Article specifies that the Service II load combination is to be used to check the service limit state requirements specified in subsequent Articles to control permanent deflections of the member after moment redistribution.

This provision is not applicable since the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges (see the Discussion of Article B6.1in this Guide).

B6.3.2 Flexure

B6.3.2.1 Adjacent to Interior-Pier Sections

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article indicate that the service limit state flange stress limits in Article 6.10.4.2 (see the Discussion of Article 6.10.4.2 in this Guide) need not be checked after moment redistribution within the regions extending in each adjacent span from interior-pier sections satisfying the requirements of Article B6.2 (see the Discussion of Article B6.2 in this Guide) to the nearest flange transition or point of dead-load contraflexure, whichever is closest. The web-bend buckling check given by Eq. 6.10.4.2.2-4 is the only check that must be satisfied within these regions. This check is to be based on the elastic moments before moment redistribution.

These provisions are not applicable since the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges (see the Discussion of Article B6.1 in this Guide).

B6.3.2.2 At All Other Locations

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article indicate that in order to control permanent deflections of the member after moment redistribution, the service limit state flange-stress limits in Article 6.10.4.2 (see the Discussion of Article 6.10.4.2 in this Guide) are only to be imposed in each adjacent span at sections outside the nearest flange transition location or point of permanent-load contraflexure, whichever is closest to the interior support under consideration. The appropriate flexural stresses due to the redistribution moments are to be added to the flexural stresses due to the Service II elastic moments prior to making these checks. The redistribution moments are to be computed according to the provisions of Article B6.3.3 (see the Discussion of Article B6.3.3 in this Guide). The composite section properties to be used in computing the stresses in the steel section and concrete deck due to the redistribution moments are also defined in this Article.

These provisions are not applicable since the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges (see the Discussion of Article B6.1in this Guide).

B6.3.3 Redistribution Moments

B6.3.3.1 At Interior-Pier Sections

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article are used to calculate the redistribution moment for the Service II loads at each interior-pier section where the Service II flange stress limits are not checked as permitted in Article B6.3.2.1 (see the Discussion of Article B6.3.2.1 in this Guide). In the approximate approach, the redistribution moment is calculated utilizing a negative-flexure effective plastic moment for the service limit state determined as specified in Article B6.5 (see the Discussion of Article B6.5 in this Guide).

These provisions are not applicable since the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges (see the Discussion of Article B6.1in this Guide).

B6.3.3.2 At All Other Locations

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article specify how to construct the Service II redistribution-moment diagram using the redistribution moments at the adjacent interior-pier sections. The redistribution-moment diagram is used to compute the Service II redistribution moments at locations other than at the interior piers. These moments are held in equilibrium by the support reactions and remain in the structure after the live loads are removed.

These provisions are not applicable since the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges (see the Discussion of Article B6.1in this Guide).

B6.4 STRENGTH LIMIT STATE

B6.4.1 Flexural Resistance

B6.4.1.1 Adjacent to Interior-Pier Sections

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provision in this Article indicates that the flexural resistances at the strength limit state of sections within the unbraced lengths immediately adjacent to interior-pier sections satisfying the requirements of Article B6.2 (see the Discussion of Article B6.2 in this Guide) from which moments are redistributed need not be checked.

This provision is not applicable since the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges (see the Discussion of Article B6.1in this Guide).

B6.4.1.2 At All Other Locations

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article indicate that sections at all other locations, i.e., at locations other than those defined in Article B6.4.1.1 (see the Discussion of Article B6.4.1.1 in this Guide) must satisfy the strength limit state provisions of Articles 6.10.7, 6.10.8.1, or A6.1, as applicable, after moment redistribution (see the Discussion of these Articles in this Guide). The appropriate redistribution moments are to be added to the factored elastic moments at the strength limit state prior to making the design checks. The redistribution moments are to be computed according to the provisions of Article B6.4.2 (see the Discussion of Article B6.4.2 in this Guide). The composite section properties to be used in computing the stresses in the steel section and concrete deck due to the redistribution moments are also defined in this Article.

These provisions are not applicable since the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges (see the Discussion of Article B6.1in this Guide).

B6.4.2 Redistribution Moments

B6.4.2.1 At Interior-Pier Sections

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article are used to calculate the redistribution moment for the strength limit state at each interior-pier section where the flexural resistance is not checked as permitted in Article B6.4.1.1 (see the Discussion of Article B6.4.1.1 in this Guide). In the approximate approach, the redistribution moment is calculated utilizing a negative-flexure effective plastic moment for the strength limit state determined as specified in Article B6.5 (see the Discussion of Article B6.5 in this Guide).

These provisions are not applicable since the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges (see the Discussion of Article B6.1in this Guide).

B6.4.2.2 At All Other Sections

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provision in this Article refers to the provisions of Article B6.3.3.2 for the procedure to construct the strength limit state redistribution-moment diagram using the redistribution moments at the adjacent interior-pier sections (see the Discussion of Article B6.3.3.2 in this Guide). The redistribution-moment diagram is used to compute the strength limit state redistribution moments at locations other than at the interior piers. These moments are held in equilibrium by the support reactions and remain in the structure after the live loads are removed.

This provision is not applicable since the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges (see the Discussion of Article B6.1in this Guide).

B6.5 EFFECTIVE PLASTIC MOMENT

B6.5.1 Interior-Pier Sections with Enhanced Moment-Rotation Characteristics

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article are used to determine the negative-flexure effective plastic moment for the calculation of the redistribution moments at the service and strength limit states (see the Discussion of Articles B6.3.3.1 and B6.4.2.1 in this Guide) for interior-pier sections satisfying the requirements of Article B6.2 (see the Discussion of Article B6.2 in this Guide) and with enhanced moment-rotation characteristics; that is, sections with transverse stiffeners spaced at $D/2$ or less over a minimum distance of $D/2$ on each side of the interior-pier section, or sections with so-called "ultracompact webs".

These provisions are not applicable since the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges (see the Discussion of Article B6.1in this Guide).

B6.5.2 All Other Interior-Pier Sections

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article are used to determine the negative-flexure effective plastic moment for the calculation of the redistribution moments at the service and strength limit states (see the Discussion of Articles B6.3.3.1 and B6.4.2.1 in this Guide) for interior-pier sections satisfying the requirements of Article B6.2 (see the Discussion of Article B6.2 in this Guide), but not satisfying the requirements of Article B6.5.1 that provide for enhanced moment-rotation characteristics (see the Discussion of Article B6.5.1 in this Guide).

These provisions are not applicable since the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges (see the Discussion of Article B6.1in this Guide).

B6.6 REFINED METHOD

B6.6.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article permit continuous-span I-section members satisfying the provisions of Article B6.2 (see the Discussion of Article B6.2 in this Guide) to alternatively be proportioned using a refined method in which a direct shakedown analysis is conducted at the service and/or strength limit states. In this analysis, the redistribution moments are determined by the simultaneous satisfaction of rotational continuity and specified inelastic moment-rotation relationships at interior-pier sections from which moments are redistributed. The elastic moment envelope due to the factored loads is used in the analysis. Sections adjacent to interior piers from which moments are redistributed are to satisfy the requirements of Article B6.3.2.1 at the service limit state and Article B6.4.1.1 at the strength limit state. Other sections are to satisfy the applicable provisions of Articles 6.10.4.2, 6.10.7, 6.10.8.1, or A6.1 after a solution is found (see the Discussion of these Articles in this Guide).

If software that handles this type of calculation along with the determination of the elastic moment envelopes does not exist, significant manual work is required in conducting the analysis calculations. The Engineer can gain some additional benefit when using a direct shakedown analysis since the restriction that interior-pier sections within the member satisfy the requirements of Article B6.2.1 (i.e., the web-slenderness requirements) is waived. Also, the directly calculated inelastic rotations at the interior-pier sections will tend to be smaller than the upper-bound values that the equations in Articles B6.3 through B6.5 are based upon (see the Discussions of Articles B6.3 through B6.5 in this Guide).

These provisions are not applicable since the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges (see the Discussion of Article B6.1in this Guide).

B6.6.2 Nominal Moment-Rotation Curves

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions in this Article provide lower-bound nominal moment-rotation curves that may be used in the refined method utilizing a direct shakedown analysis (see the Discussion of Article B6.6.1 in this Guide) at interior-pier sections satisfying the requirements of Article B6.2 (see the Discussion of Article B6.2 in this Guide). Curves are provided for interior-pier sections satisfying the requirements of Article B6.5.1 to provide enhanced moment-rotation characteristics (see the Discussion of Article B6.5.1 in this Guide), and for other interior-pier sections not satisfying those requirements. Interior-pier sections not satisfying the requirements of Article B6.2 are assumed to remain elastic in the analysis, and are to satisfy the provisions of Articles 6.10.4.2, 6.10.8.1, or Article A6.1, as applicable, after a solution is found (see the Discussion of these Articles in this Guide).

These provisions are not applicable since the use of moment redistribution methods has been specifically excluded from the scope of this Guide for the design of routine steel I-girder bridges (see the Discussion of Article B6.1in this Guide).

APPENDIX C6 BASIC STEPS FOR STEEL BRIDGE SUPERSTRUCTURES

C6.1 GENERAL

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

This Article serves as an introduction to an outline contained in Articles C6.2 and C6.3 (see the Discussion of Articles C6.2 and C6.3 in this Guide) that provides a generic overview of the design process for steel bridges. The outline is not fully complete and should not be used as a substitute for a working knowledge of the provisions of Section 6 of the AASHTO LRFD BDS, but much of this outline can still be helpful in providing the basic design steps for the routine steel I-girder bridges covered by this Guide. For each design step, the outline refers to the specific Article(s) within Section 6 that contain the design provisions relevant to that step. This outline was used in the development of the Design Task Quick Links provided with this Guide.

C6.2 GENERAL CONSIDERATIONS

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The portions of the outline described in Article C6.1 (see the Discussion of Article C6.1 in this Guide) dealing with general considerations such as the general design philosophy, limit states, and design and location features are provided in this Article and are considered applicable to the routine steel I-girder bridges covered by this Guide.

C6.3 SUPERSTRUCTURE DESIGN

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

The portions of the outline described in Article C6.1 (see the Discussion of Article C6.1 in this Guide) dealing with steel superstructure design are provided in this Article and are considered partially applicable as only the design steps pertinent to the design of routine steel I-girder bridges covered by this Guide are applicable.

Note that the Design Tasks Quick Links provided near the beginning of this Guide are based directly on the outline presented in this Article.

C6.4 FLOWCHARTS FOR FLEXURAL DESIGN OF I-SECTION MEMBERS

C6.4.1 Flowchart for LRFD Article 6.10.3

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The flowchart provided in this Article is helpful to guide the Engineer through the provisions of Article 6.10.3 dealing with the design for constructibility (see the Discussion of Article 6.10.3 in this Guide). This flowchart is applicable to the routine steel I-girder bridges covered by this Guide and is strongly recommended for use in conjunction with this Guide.

C6.4.2 Flowchart for LRFD Article 6.10.4

Determination of applicability, *All Routine Steel I-girder Bridges*: Partially applicable.

Discussion:

The flowchart provided in this Article is helpful to guide the Engineer through the provisions of Article 6.10.4 dealing with the design for the service limit state (see the Discussion of Article 6.10.4 in this Guide) and is strongly recommended for use in conjunction with this Guide. The portions of the flowchart dealing with optional moment redistribution and shored construction (see the Discussion of Articles B6.1 and Article 6.10.1.1.1a in this Guide) are not applicable to the routine steel I-girder bridges covered by this Guide.

C6.4.3 Flowchart for LRFD Article 6.10.5

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The flowchart provided in this Article is helpful to guide the Engineer through the provisions of Articles 6.5.3 and 6.6 dealing with the design for the fatigue and fracture limit state (see the Discussion of Articles 6.5.3 and 6.6 in this Guide). This flowchart is applicable to the routine steel I-girder bridges covered by this Guide and is strongly recommended for use in conjunction with this Guide.

C6.4.4 Flowchart for LRFD Article 6.10.6

Determination of applicability, *All Routine Steel I-girder Bridges*: Applicable.

Discussion:

The flowchart provided in this Article is helpful to guide the Engineer through the provisions of Article 6.10.6 dealing with the design for the strength limit state (see the Discussion of Article 6.10.6 in this Guide) and is strongly recommended for use in conjunction with this Guide. The portion of the flowchart dealing with optional moment redistribution (see the Discussion of Article B6.1 in this Guide) is not applicable to the routine steel I-girder bridges covered by this Guide. Only the portion of the flowchart dealing with the design of composite sections in positive flexure is applicable to the design of routine simple span I-girder bridges covered by this Guide.

C6.4.5 Flowchart for LRFD Article 6.10.7

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Applicable.

Discussion:

The flowchart provided in this Article is helpful to guide the Engineer through the provisions of Article 6.10.7 dealing with the design of composite sections in positive flexure at the strength limit state (see the Discussion of Article 6.10.7 in this Guide). This flowchart is applicable to the routine steel I-girder bridges covered by this Guide, except as noted below for simple span bridges, and is strongly recommended for use in conjunction with this Guide.

Simple Span Bridges:

The portion of the flowchart dealing with continuous spans is not applicable.

C6.4.6 Flowchart for LRFD Article 6.10.8

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

The flowchart provided in this Article is helpful to guide the Engineer through the provisions of Article 6.10.8 dealing with the design of composite sections in negative flexure at the strength limit state and noncomposite sections subject to positive or negative flexure at the strength limit state and during construction (see the Discussion of Articles 6.10.8 and 6.10.3 in this Guide), and is strongly recommended for use in conjunction with this Guide. Article 6.10.8 assumes the section under consideration is a slender web section or is conservatively treated as a slender web section (see the Commentary for Article 6.10.6.2.3 and the Discussion of Article 6.10.6.2.3 in this Guide for further discussion on the definition and categorization of compact web, noncompact web, and slender web sections). The applicability of this flowchart is discussed further below.

Simple Span Bridges:

The portions of the flowchart dealing with the determination of F_{nc} for the discretely braced top (compression) flange are applicable for the routine simple span I-girder bridges covered by this Guide when checking the noncomposite section during construction using Eq. 6.10.3.2.1-2 (see the Discussion of Article 6.10.3.2.1 in this Guide) if the section is a slender web section or is conservatively treated as a slender web section (which is not recommended for rolled-beam sections in particular). This flowchart is not applicable to these bridges at the strength limit state as simple spans are subject to positive flexure only.

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

This flowchart is applicable for the routine multi-span continuous rolled-beam and plate girder bridges covered by this Guide at the strength limit state if the sections in negative flexure are slender web sections or are conservatively treated as slender web sections (which is not recommended for rolled-beam sections but may be reasonable for plate girder sections). The portions of the flowchart dealing with the determination of F_{nc} of discretely braced compression flanges are also applicable for these bridges when checking the noncomposite section during construction using Eq. 6.10.3.2.1-2 (see the Discussion of Article 6.10.3.2.1 in this Guide).

C6.4.7 Flowchart for Appendix A6

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The flowchart provided in this Article is helpful to guide the Engineer through the provisions of the optional Appendix A6 dealing with the design of composite sections in negative flexure at the strength limit state and noncomposite sections subject to positive or negative flexure at the strength limit state, and in some cases, during construction (see the Discussion of Article Appendix A6 in this Guide), and is strongly recommended for use in conjunction with this Guide. Appendix A6 may only be used for sections that satisfy the restrictions specified in Article 6.10.6.2.3 (see the Discussion of Article 6.10.6.2.3 in this Guide). The applicability of this flowchart is discussed further below.

Simple Span Bridges:

The portions of the flowchart dealing with the determination of M_{nc} for lateral-torsional buckling of the discretely braced top (compression) flange are applicable for the routine simple span I-girder bridges covered by this Guide with nonslender webs when the provisions of Article A6.3.3 are used to compute M_{nc} to account for the beneficial effect of the St. Venant torsional constant, J, when checking the noncomposite section during construction using Eq. 6.10.3.2.1-2 (see the Discussion of Article 6.10.3.2.1 in this Guide), which is recommended for rolled-beam bridges in particular. This Article is not applicable to these bridges at the strength limit state as simple spans are subject to positive flexure only.

Multi-span Continuous Rolled Beam Bridges:

This flowchart is applicable for the routine multi-span continuous rolled beam bridges covered by this Guide at the strength limit state if the sections in negative flexure satisfy the restrictions specified in Article 6.10.6.2.3 for the use of Appendix A6, which is typically the case for rolled-beam sections. The portions of the flowchart dealing with the determination of M_{nc} for lateral-torsional buckling of discretely braced compression flanges are applicable for bridges with nonslender webs when the provisions of Article A6.3.3 are used to compute M_{nc} to account for the beneficial effect of the St. Venant torsional constant, J, when checking the noncomposite section during construction using Eq. 6.10.3.2.1-2 (see the Discussion of Article 6.10.3.2.1 in this Guide), which is recommended for rolled-beam bridges.

Multi-span Continuous Plate Girder Bridges:

This flowchart is applicable for the routine multi-span continuous plate girder bridges covered by this Guide at the strength limit state if the sections in negative flexure satisfy the restrictions specified in Article 6.10.6.2.3 for the use of Appendix A6. The portions of the flowchart dealing with the determination of M_{nc} for lateral-torsional buckling of discretely braced compression flanges are applicable for bridges with nonslender webs when the provisions of Article A6.3.3 are used to compute M_{nc} to account for the beneficial effect of the St. Venant torsional constant, J, when checking the noncomposite section during construction using Eq. 6.10.3.2.1-2 (see the Discussion of Article 6.10.3.2.1 in this Guide).

C6.4.8 Flowchart for Article D6.4.1

Determination of applicability, *Simple Span Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

The flowchart provided in this Article is helpful to guide the Engineer through the provisions of Article D6.4.1, which should always be invoked when calculating the nominal lateral-torsional buckling resistance of a prismatic unbraced length for uniform bending according to the provisions of Articles 6.10.8.2.3a and 6.10.8.2.3b, and when a moment-gradient modifier, C_b, is calculated for the unbraced length under consideration and is greater than 1.0 (see the Discussions of Articles 6.10.8.2.3a, 6.10.8.2.3b, and D6.4.1 in this Guide). The provisions of this Article can result in the nominal flexural resistance of the compression flange reaching the plateau strength (F_{max}) for lateral-torsional buckling at significantly larger unbraced lengths under moment-gradient conditions when the effects of the moment-gradient are included in determining the limits on the unbraced length, L_b (see the dashed curve in Figure C6.10.8.2.1-1). The applicability of this flowchart is discussed further below.

Simple Span Bridges:

This flowchart is not applicable for the routine simple span bridges covered by this Guide at the strength limit state because the top flanges are continuously braced. This flowchart may be applicable for these bridges when computing the lateral-torsional buckling resistance of a prismatic unbraced length of a discretely braced top (compression) flange of the noncomposite section during construction using the provisions of Articles 6.10.8.2.3a and 6.10.8.2.3b for use in Eq. 6.10.3.2.1-2 (see the Discussion of Articles 6.10.3.2.1, 6.10.8.2.3a, and 6.10.8.2.3b in this Guide), which is not recommended for rolled-beam bridges in particular, and when C_b is calculated for the unbraced length under consideration and is greater than 1.0. Note however that C_b is typically taken equal to 1.0 when checking lateral-torsional buckling of the critical noncomposite section in regions of positive flexure during construction; otherwise, use Eq. 6.10.8.2.3b-1 to compute C_b (see the Discussion of Article 6.10.8.2.3b in this Guide).

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

This flowchart is applicable for the routine multi-span continuous rolled-beam and plate girder bridges covered by this Guide at the strength limit state when computing the lateral-torsional buckling resistance of a prismatic unbraced length of a discretely braced bottom (compression) flange in regions of negative flexure using the provisions of Articles 6.10.8.2.3a and 6.10.8.2.3b (which is not recommended for rolled-beam bridges), and when C_b is calculated for the unbraced length under consideration and is greater than 1.0. This flowchart is also applicable for these bridges when computing the lateral-torsional buckling resistance of a prismatic unbraced length of the discretely braced compression flanges of the noncomposite section during construction using the provisions of Articles 6.10.8.2.3a and 6.10.8.2.3b for use in Eq. 6.10.3.2.1-2 (which is not recommended for rolled-beam bridges, see the Discussion of Article 6.10.3.2.1 in this Guide), and when C_b is calculated for the unbraced length under consideration and is greater than 1.0. Note however that C_b is typically taken equal to 1.0 when checking lateral-torsional buckling of the critical noncomposite section in regions of positive flexure during construction; otherwise, use Eq. 6.10.8.2.3b-1 to compute C_b (see the Discussion of Article 6.10.8.2.3b in this Guide).

C6.4.9 Flowchart for Article D6.4.2

Determination of applicability, *All Routine Steel I-girder Bridges*:Conditionally applicable.

Discussion:

The flowchart provided in this Article is helpful to guide the Engineer through the provisions of Article D6.4.2, which should always be invoked when calculating the nominal lateral-torsional buckling resistance of a prismatic unbraced length for uniform bending according to the provisions of Articles A6.3.3.1 and A6.3.3.2 in the optional Appendix A6, and when a moment-gradient modifier, C_b, is calculated for the unbraced length under consideration and is greater than 1.0 (see the Discussions of Articles A6.3.3.1, A6.3.3.2, and D6.4.2 in this Guide). The provisions of this Article can result in the nominal flexural resistance of the compression flange reaching the plateau strength (M_{max}) for lateral-torsional buckling at significantly larger unbraced lengths under moment-gradient conditions when the effects of the moment-gradient are included in determining the limits on the unbraced length, L_b (see the dashed curve in Figure C6.10.8.2.1-1). The applicability of this flowchart is discussed further below.

Simple Span Bridges:

This flowchart is not applicable for the routine simple span bridges covered by this Guide at the strength limit state because the top flanges are continuously braced. This flowchart may be applicable for these bridges when computing the lateral-torsional buckling resistance of a prismatic unbraced length of noncomposite sections with nonslender webs during construction for use in Eq. 6.10.3.2.1-2 (see the Discussion of Article 6.10.3.2.1 in this Guide) using the provisions of Articles A6.3.3.1 and A6.3.3.2 to account for the beneficial effect of the St. Venant torsional constant, J, which is recommended for rolled-beam bridges in particular, and when C_b is calculated for the unbraced length under consideration and is greater than 1.0. Note however that C_b is typically taken equal to 1.0 when checking lateral-torsional buckling of the critical noncomposite section in regions of positive flexure during construction; otherwise, use Eq. 6.10.8.2.3b-1 to compute C_b (see the Discussion of Article 6.10.8.2.3b in this Guide).

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

This flowchart is applicable for the routine multi-span continuous rolled-beam and plate girder bridges covered by this Guide at the strength limit state when computing the lateral-torsional buckling resistance of a prismatic unbraced length of sections in regions of negative flexure with nonslender webs using the provisions of Articles A6.3.3.1 and A6.3.3.2 (which is recommended for rolled-beam bridges), and when C_b is calculated for the unbraced length under consideration and is greater than 1.0. This flowchart is also applicable for these bridges when computing the lateral-torsional buckling resistance of a prismatic unbraced length of noncomposite sections with nonslender webs during construction for use in Eq. 6.10.3.2.1-2 (see the Discussion of Article 6.10.3.2.1 in this Guide) using the provisions of Articles A6.3.3.1 and A6.3.3.2 to account for the beneficial effect of the St. Venant torsional constant, J (which is recommended for rolled-beam bridges), and when C_b is calculated for the unbraced length under consideration and is greater than 1.0. Note however that C_b is typically taken equal to 1.0 when checking lateral-torsional buckling of the critical noncomposite section in regions of positive flexure during construction; otherwise, use Eq. 6.10.8.2.3b-1 to compute C_b (see the Discussion of Article 6.10.8.2.3b in this Guide).

C6.5 FLOWCHARTS FOR LRFD ARTICLES 6.9.4 AND 6.12.2.2.2

C6.5.1 Flowchart for LRFD Article 6.9.4

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The flowchart provided in this Article is helpful to guide the Engineer through the provisions of Article 6.9.4 dealing with the determination of the nominal compressive resistance of noncomposite I- and box-section members (see the Discussion of Article 6.9.4 in this Guide).

This flowchart is not applicable to the routine steel I-girder bridges covered by this Guide as the members in these bridges are not fully noncomposite, are not box-section members, and are subject to flexure only.

C6.5.2 Flowchart for LRFD Article 6.12.2.2.2

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The flowchart provided in this Article is helpful to guide the Engineer through the provisions of Article 6.12.2.2.2 dealing with the determination of the nominal flexural resistance of rectangular noncomposite box-section members (see the Discussion of Article 6.12.2.2.2 in this Guide). This flowchart is not applicable to the routine steel I-girder bridges covered by this Guide as noncomposite box-section members are not used in these bridges.

APPENDIX D6 FUNDAMENTAL CALCULATIONS FOR FLEXURAL MEMBERS

D6.1 PLASTIC MOMENT

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The plastic moment, M_p, is defined in the AASHTO LRFD BDS as the resisting moment about the major axis of a fully yielded cross-section. M_p is used as a theoretical measure of the maximum potential flexural resistance at the strength limit state of a noncomposite or composite section satisfying specific steel grade, flange and web slenderness, compression-flange bracing and ductility requirements, as applicable. For sections that can achieve the full plastic-moment resistance, it is assumed that the section is completely elastic up to M_p and then rotates inelastically at M_p with no increase in the moment resistance. The effects of strain hardening are conservatively ignored. This idealized moment-rotation behavior is termed elastic-perfectly-plastic behavior. In the AASHTO LRFD BDS, composite sections in straight bridges in regions of positive flexure that can achieve flexural resistances at or near M_p are termed compact sections (see the Discussion of Article 6.10.6.2.2 in this Guide). Composite sections in regions of negative flexure and noncomposite sections subject to positive or negative flexure in straight bridges that can achieve flexural resistances of M_p are termed compact web sections and are less commonly used (see the Commentary for Article 6.10.6.2.3 and the Discussion of Article 6.10.6.2.3 in this Guide for further discussion on the definition and categorization of compact web, noncompact web, and slender web sections).

M_p is calculated as the moment of the plastic forces acting on the cross-section about the plastic neutral axis (*PNA*). For sections subject to flexure only, M_p may be calculated as the moment of the plastic forces about any axis parallel to the *PNA*. Plastic forces in steel portions of the cross-section are calculated using the yield strengths of the flanges, web, and longitudinal reinforcing steel, as appropriate. Plastic forces in concrete portions of the cross-section (in compression only) are based on a rectangular stress block, with the magnitude of the compressive stress taken equal to $0.85f'_c$. Concrete in tension is neglected. Equations to calculate these plastic forces are given in this Article. The position of the *PNA* is calculated based on the equilibrium condition that there is no net axial force acting on the cross-section.

For composite sections, the stress distribution in the cross-section at M_p is assumed independent of the manner in which the stresses are induced into the beam. Also, creep and shrinkage are assumed to have no effect on the internal stress distribution at M_p. Thus, when checking the flexural resistance of a composite section against M_p, the moments acting on the non-composite, long-term composite and short-term composite sections may be directly summed for comparison to M_p. The effect of the sequence of application of the different types of loads on the stress states and partial yielding within the cross-section on the resistance is not considered. For composite sections in positive flexure, the attainment of M_p is possible only if the steel girder is provided with an adequate number of shear connectors so that the horizontal shear force from the concrete deck is effectively transmitted to the steel girder (see the Discussion of Article 6.10.10.4 in this Guide). The natural bond between the steel and concrete is not sufficient by itself. M_p for a composite section in positive flexure can be determined as follows:

- Calculate the plastic forces of each individual component in the cross-section and use them to determine whether the *PNA* is in the web, top flange or concrete deck;
- Calculate the location of the *PNA* within the element determined in Step 1; and
- Calculate M_p. Table D6.1-1 provides equations for the location of the *PNA* and for calculating M_p for seven possible conditions depending on the location of the *PNA*.

In Table D6.1-1, d is the distance from the element plastic force to the *PNA*. The element forces are assumed to act at the mid-thickness of the flanges and concrete deck, at the mid-depth of the web and at the center of the longitudinal reinforcement. All element forces, dimensions, and distances are to be taken as positive. The conditions should be checked in the order listed in the table. The forces in the longitudinal reinforcement may be conservatively neglected by setting the terms, P_{rb} and P_{rt}, equal to zero in the equations given in the table.

For composite sections in negative flexure, a similar procedure can be used. In this case, however, the tensile strength of the concrete is ignored, and the contribution of the longitudinal reinforcement should be included. Table D6.1-2 contains the equations for the two cases most likely to occur in practice. Again, the conditions should be checked in the order listed in the table.

For homogenous doubly symmetric noncomposite sections, M_p may simply be calculated as F_yZ, where Z is the plastic section modulus calculated as the sum of the first moments of the flange and web areas about the *PNA*. For rolled wide-flange sections, values of Z are tabulated in the AISC *Manual of Steel Construction*. The plastic moment of a noncomposite section may also be calculated by simply eliminating the terms pertaining to the concrete deck and longitudinal reinforcement from the equations in Tables D6.1-1 and D6.1-2, as applicable.

For further information on the plastic moment and example calculations of the plastic moment, consult Section 6.4.5.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

Simple Span Bridges:

These provisions are used for the routine simple span bridges covered by this Guide to compute the plastic moment for the composite section, which is necessary to determine the nominal flexural resistance at the strength limit state if the section qualifies and is treated as a compact section, which is typically the case. These provisions may also be used for these bridges to compute the plastic moment for the noncomposite section if the section has a compact or noncompact web and the provisions of Article A6.3.3 are used to optionally determine the nominal lateral-torsional buckling resistance for use in checking Eq. 6.10.3.2.1-2 during construction in order to include the beneficial effect of the St. Venant torsional constant, *J* (see the Discussion of Articles A6.3.3 and 6.10.3.2.1 in this Guide).

Multi-span Continuous Rolled Beam Bridges:

These provisions are used for the routine multi-span continuous rolled beam bridges covered by this Guide to compute the plastic moment for the composite section in regions of positive flexure, which is necessary to determine the nominal flexural resistance at the strength limit state if the section qualifies and is treated as a compact section, which will typically be the case. These provisions may also be used for these bridges to compute the plastic moment for the composite section or for the noncomposite section (if no shear studs are used) in regions of negative flexure if the section satisfies the restrictions specified in Article 6.10.6.2.3 (see the Discussion of Article 6.10.6.2.3 in this Guide) and the optional provisions of Appendix A6 are used to determine the nominal flexural resistance, which will typically be the case and is strongly encouraged for rolled-beam sections. These provisions may also be used for these bridges to compute the plastic moment for the noncomposite section if the section has a compact or noncompact web and the provisions of Article A6.3.3 are optionally used to determine the nominal lateral-torsional buckling resistance for use in checking Eq. 6.10.3.2.1-2 during construction in order to include the beneficial effect of the St. Venant torsional constant, *J* (see the Discussion of Articles A6.3.3 and 6.10.3.2.1 in this Guide).

Multi-span Continuous Plate Girder Bridges:

These provisions are used for the routine multi-span continuous plate girder bridges covered by this Guide to compute the plastic moment for the composite section in regions of positive flexure, which is necessary to determine the nominal flexural resistance at the strength limit state if the section qualifies and is treated as a compact section, which will typically be the case. These provisions may also be used for these bridges to compute the plastic moment for the composite section in regions of negative flexure or for the noncomposite section (if no shear studs are used) if the section satisfies the restrictions specified in Article 6.10.6.2.3 (see the Discussion of Article 6.10.6.2.3 in this Guide) and the optional provisions of Appendix A6 are used to determine the nominal flexural resistance at the strength limit state. These provisions may also be used for these bridges to compute the plastic moment for the noncomposite section if the section has a compact or noncompact web and the provisions of Article A6.3.3 are optionally used to determine the nominal lateral-torsional buckling resistance for use in checking Eq. 6.10.3.2.1-2 during construction in order to include the beneficial effect of the St. Venant torsional constant, *J* (see the Discussion of Articles A6.3.3 and 6.10.3.2.1 in this Guide).

D6.2 YIELD MOMENT

D6.2.1 Noncomposite Sections

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The yield moment, M_y, is defined in the AASHTO LRFD BDS as the moment at which an outer fiber, in a member subjected to flexure about the major-axis, attains the nominal yield stress neglecting the effect of any residual stresses.

For a noncomposite section, this Article states that M_y is to be taken as the smaller of the moment required to cause nominal first yielding in the compression flange, M_{yc}, or the moment required to cause nominal first yielding in the tension flange, M_{yt}, at the strength limit state.

The ratio of M_p/M_y is a property of the cross-sectional shape known as the shape factor. For doubly symmetric noncomposite I-shapes bent about their major axis, the shape factor is approximately 1.12.

The computation of M_y for a noncomposite section may be required for the routine steel I-girder bridges covered by this Guide if the section has a compact or noncompact web and the provisions of Article A6.3.3 are optionally used to determine the nominal lateral-torsional buckling resistance for use in checking Eq. 6.10.3.2.1-2 during construction in order to include the beneficial effect of the St. Venant torsional constant, J (see the Discussion of Articles A6.3.3 and 6.10.3.2.1 in this Guide). The computation of M_y for a noncomposite section in regions of negative flexure (i.e., M_{yc} and M_{yt}) if no shear studs are used may also be required at the strength limit state if the noncomposite section satisfies the restrictions specified in Article 6.10.6.2.3 (see the Discussion of Article 6.10.6.2.3 in this Guide) and the optional provisions of Appendix A6 are used to compute the nominal flexural resistance. See the Commentary for Article 6.10.6.2.3 and the Discussion of Article 6.10.6.2.3 in this Guide for further discussion on the definition and categorization of compact web, noncompact web, and slender web sections.

For further information on the yield moment and example calculations of the yield moment, consult Section 6.4.5.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

D6.2.2 Composite Sections in Positive Flexure

Determination of applicability, *Simple Span Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Applicable.

Discussion:

The yield moment, M_y, is defined in the AASHTO LRFD BDS as the moment at which an outer fiber, in a member subjected to flexure about the major-axis, attains the nominal yield stress neglecting the effect of any residual stresses.

For composite sections in positive flexure, M_y is to be taken as the sum of the moments applied separately to the steel, short-term and long-term composite sections to cause nominal first yielding in either flange at the strength limit state. M_y is taken as the smaller of the moment required to cause nominal first yielding in the compression flange, M_{yc}, or the moment required to cause nominal first yielding in the tension flange, M_{yt}.

In a composite girder, moments are applied to different sections and this fact must be appropriately accounted for in the computation of M_y. M_y for a composite section in positive flexure can therefore be determined as follows: 1) calculate the moment, M_{D1}, caused by the factored permanent load applied before the concrete deck has hardened or is made composite and apply this moment to the steel section; 2) calculate the moment, M_{D2}, caused by the remainder of the factored permanent load and apply this moment to the long-term composite section; 3) calculate the additional moment, M_{AD}, that must be applied to the short-term composite section to cause nominal yielding in either steel flange; and 4) calculate M_y as the sum of the total permanent load moment and M_{AD} (see Eqs. D6.2.2-1 and D6.2.2-2 and the Discussion of Article 6.10.1.1.1b in this Guide). In regions of positive flexure, the longitudinal reinforcement may be neglected in the calculation of S_{ST} and S_{LT} in Eq. D6.2.2-1.

The ratio of M_p/M_y is a property of the cross-sectional shape known as the shape factor. For singly symmetric composite girders in regions of positive flexure, values of the shape factor on the order of 1.4 to 1.6 are quite common.

For further information on the yield moment and example calculations of the yield moment, consult Section 6.4.5.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

Simple Span Bridges:

The computation of M_y for a composite section in positive flexure is not applicable to the routine steel simple span bridges covered by this Guide, as M_y is not required for the determination of the nominal flexural resistance of a simple-span bridge at the strength limit state (see the Discussion of Article 6.10.7.1.2 in this Guide).

Multi-span Continuous Rolled Beam Bridges and Multi-span Continuous Plate Girder Bridges:

The computation of M_y for a composite section in positive flexure is applicable to the routine steel multi-span continuous rolled beam and plate girder bridges covered by this Guide for the determination of the nominal flexural resistance at the strength limit state if the section qualifies and is treated as a compact section, which will typically be the case (see the Discussion of Articles 6.10.6.2.2 and 6.10.7.1.2 in this Guide).

D6.2.3 Composite Sections in Negative Flexure

Determination of applicability, *Simple Span Bridges*: Not applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

The yield moment, M_y, is defined in the AASHTO LRFD BDS as the moment at which an outer fiber, in a member subjected to flexure about the major-axis, attains the nominal yield stress neglecting the effect of any residual stresses.

For composite sections in negative flexure, the procedure specified for the calculation of M_y (including M_{yc} and M_{yt}) for composite sections in positive flexure in Article D6.2.2 is followed, except the composite section for both the short-term and long-term moments applied to the composite section is to consist of the steel section and the longitudinal reinforcement within the effective width of the concrete deck. Thus, S_{ST} and S_{LT} in Eq. D6.2.2-1 are taken as the same value (see Eqs. D6.2.2-1 and D6.2.2-2 and the Discussion of Articles D6.2.2 and 6.10.1.1.1c in this Guide). Also, M_{yt} is to be taken with respect to either the tension flange or the longitudinal reinforcement, whichever yields first. For the calculation of M_{yt} with respect to the longitudinal reinforcement, M_{D1} is to be taken equal to zero in Eqs. D6.2.2-1 and D6.2.2-2, and F_{yf} in Eq. D6.2.2-1 is to be taken equal to the specified minimum yield strength of the longitudinal reinforcement.

For further information on the yield moment and example calculations of the yield moment, consult Section 6.4.5.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

Simple Span Bridges:

The computation of M_y for a composite section in negative flexure is not applicable to the routine steel simple span bridges covered by this Guide, as simple-span bridges are not subject to negative flexure.

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

The computation of M_y for a composite section in negative flexure (i.e., M_{yc} and M_{yt}) is applicable to the routine steel multi-span continuous rolled beam and plate girder bridges covered by this Guide if the section satisfies the restrictions specified in Article 6.10.6.2.3 (see the Discussion of Article 6.10.6.2.3 in this Guide) and the optional provisions of Appendix A6 are used to compute the nominal flexural resistance at the strength limit state, which will typically be the case and is strongly encouraged for rolled-beam sections, and which may be possible for some plate-girder sections.

D6.2.4 Sections with Cover Plates

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The yield moment, M_y, is defined in the AASHTO LRFD BDS as the moment at which an outer fiber, in a member subjected to flexure about the major-axis, attains the nominal yield stress neglecting the effect of any residual stresses.

For composite sections in positive or negative flexure, M_y is to be taken as the sum of the moments applied separately to the steel, short-term and long-term composite sections to cause nominal first yielding in either flange at the strength limit state. For both noncomposite and composite sections, M_y is taken as the smaller of the moment required to cause nominal first yielding in the compression flange, M_{yc}, or the moment required to cause nominal first yielding in the tension flange, M_{yt}. See the Discussion of Articles D6.2.1 through D6.2.3 in this Guide for further information on the computation of M_y for noncomposite and composite sections.

For sections containing flange cover plates, M_{yc} or M_{yt} is to be taken as the smallest value of moment associated with nominal first yielding based on the stress in either the flange under consideration or in any of the cover plates attached to that flange, whichever yields first.

The provisions of this Article are not applicable to the routine steel I-girder bridges covered by this Guide as cover plates are not used, nor are they recommended for use, on these bridges.

For further information on the yield moment and example calculations of the yield moment, consult Section 6.4.5.3 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

D6.3 DEPTH OF THE WEB IN COMPRESSION

D6.3.1 In the Elastic Range (D_c)

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Applicable.

Discussion:

The depth of the web in compression in the elastic range, D_c, is used primarily in computing the web bend-buckling resistance, F_{crw}, and the web load-shedding factor, R_b (see the Discussion of Articles 6.10.1.9.1 and 6.10.1.10.2 in this Guide). For composite sections in negative flexure and noncomposite sections subject to positive or negative flexure, D_c is also used to determine whether the section qualifies as a slender or a nonslender web section for determining the nominal flexural resistance (see the Discussion of Article 6.10.6.2.3 in this Guide).

D_c for composite sections is a function of the dead-to-live load stress ratio in the elastic range of stress at the service, fatigue, and strength limit states. This is because in a composite girder, the dead and live loads are applied to different sections. This is an especially important consideration for composite sections since the dead-load stress has a significant effect on the location of the elastic neutral axis. Note that when checking the section for web bend-buckling during construction, however, while the girder is still in the noncomposite condition before the concrete deck hardens, D_c of the steel section alone, which is a section property independent of the stress, is always to be used in the calculations.

D_c of the composite section at sections in positive flexure increases with increasing span length because of the increasing dead-to-live load ratio. With increasing spans, the larger noncomposite dead load stresses acting on the steel section alone effectively cause the neutral axis to be much lower than it would be if all loads were applied to the composite section, which obviously increases the depth of the web in compression. Therefore, where applicable, it is important to recognize the effect of the dead load stress on the location of the neutral axis at these sections. This Article states that D_c for composite sections in positive flexure is to be taken as the depth over which the algebraic sum of the stresses acting on the steel, long-term composite and short-term composite sections due to the dead and live loads, plus impact, is compressive; Eq. D6.3.1-1 may be used to calculate D_c at such sections. However, according to the AASHTO LRFD BDS, for composite sections in positive flexure at the service and strength limit states, D_c only needs to be employed in the computation of the nominal flexural resistance for sections in which longitudinal web stiffeners are required (reasons for this are discussed further in the Commentary for Article 6.10.1.9.1). Therefore, the computation of D_c is not applicable (or necessary) at the service and strength limit states for composite sections in positive flexure in the routine steel I-girder bridges covered by this Guide, which do not have longitudinal web stiffeners.

The concrete deck is typically not considered to be effective in tension for composite sections in negative flexure, except perhaps at the fatigue and service limit states as permitted when certain conditions are satisfied (see the Discussion of Articles 6.6.1.2.1 and 6.10.4.2.1 in this Guide). The distance between the neutral-axis locations for the steel and composite sections is small when the concrete deck is not considered effective, as the composite section only consists of the steel section plus the longitudinal reinforcement. As a result, the location of the neutral axis for the composite section is essentially unaffected by the dead load stress. In fact, accounting for the effect of the dead load stress results in a smaller value of D_c in regions of negative flexure.

Therefore, for the majority of situations involving composite sections in negative flexure, this Article conservatively specifies the use of D_c computed for the section consisting of the steel girder plus the longitudinal reinforcement only, without considering the algebraic sum of the stresses acting on the noncomposite and composite sections.

A single exception to the preceding requirement occurs if the concrete deck is assumed effective in tension in regions of negative flexure at the service limit state, as permitted for composite girders satisfying the conditions specified in Article 6.10.4.2.1 (see the Discussion of Article 6.10.4.2.1 in this Guide); in such cases, Eq. D6.3.1-1 *must* be used to compute D_c. For this case, in Figure D6.3.1-1, the stresses f_c and f_t should be switched, the signs shown in the stress diagram should be reversed, t_{fc} should be the thickness of the bottom flange, and D_c should instead extend from the neutral axis down to the top of the bottom flange. When calculating the web bend-buckling resistance, F_{crw}, at the service limit state (see the Discussion of Article 6.10.4.2.2 in this

Guide), a more precise calculation of D_c, accounting for the beneficial effect of the dead load stress in this case, is required for the composite section whenever the concrete deck is permitted to be considered effective in tension. Otherwise, the reduction in F_{crw} will be too large and not reflective of the actual potential web bend-buckling resistance at this limit state.

For further information on the elastic depth of the web in compression, D_c, and example calculations of D_c, consult Section 6.4.5.4.1 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

Simple Span Bridges:

For the routine simple span bridges covered by this Guide, only D_c for the noncomposite steel section must be calculated to check the web bend-buckling resistance of the section during construction (see the Discussion of Article 6.10.3.2.1 in this Guide) and to determine whether the web of the noncomposite steel section is slender or nonslender (see the Discussion of Article 6.10.6.2.3 in this Guide). D_c need not be calculated at the service or strength limit states.

Multi-span Continuous Rolled Beam Bridges:

For the routine multi-span continuous rolled beam bridges covered by this Guide, D_c for the noncomposite steel section must be calculated to check the web bend-buckling resistance of the section during construction (see the Discussion of Article 6.10.3.2.1 in this Guide) and to determine whether the web of the noncomposite steel section is slender or nonslender (see the Discussion of Article 6.10.6.2.3 in this Guide). D_c need not be calculated at the service or strength limit states for composite sections in regions of positive flexure.

For composite sections and noncomposite sections (if shear studs are not provided) in regions of negative flexure at the strength limit state, D_c must be calculated to determine whether the section is a slender or nonslender web section (rolled beams are typically nonslender web sections). In this case, for composite sections, D_c is to be computed for the section consisting of the steel girder plus the longitudinal reinforcement only, and for noncomposite sections, D_c is to be computed for the steel section only. For composite sections and noncomposite sections in these regions at the service limit state, D_c must be calculated to check the web bend-buckling resistance of the section (see the Discussion of Article 6.10.4.2.2 in this Guide). In this case, for composite sections, if the requirements of Article 6.10.4.2.1 are satisfied and the concrete deck is permitted to be assumed effective in tension (see the Discussion of Article 6.10.4.2.1 in this Guide), Eq. D6.3.1-1 *must* be used to compute D_c. Otherwise, D_c is to be computed for the section consisting of the steel girder plus the longitudinal reinforcement only. For noncomposite sections, D_c is always to be computed for the steel section only.

Multi-span Continuous Plate Girder Bridges:

See the preceding Discussion for Multi-span Continuous Rolled Beam Bridges. In addition, for slender web composite sections and noncomposite sections (if shear studs are not provided) in regions of negative flexure at the strength limit state, D_c must be calculated to determine the web load-shedding factor, R_b (see the Discussion of Article 6.10.1.10.2 in this Guide).

D6.3.2 At Plastic Moment (D_{cp})

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The depth of the web in compression at the plastic moment, D_{cp}, is used primarily to determine if a composite section in regions of positive flexure qualifies as a compact section at the strength limit state (see the Discussion of Article 6.10.6.2.2 in this Guide), and to determine if a nonslender, composite section or a nonslender, noncomposite section (if shear studs are not provided) in regions of negative flexure qualifies as either a compact web or a noncompact web section at the strength limit state (see the Discussion of Article A6.2.1 in this Guide).

At sections in positive flexure, D_{cp} is typically smaller than the elastic depth of the web in compression, D_c, as plastic strains associated with moments larger than M_y are incurred. In fact, for composite sections in positive flexure, the neutral axis at the plastic moment, M_p, will most often be located either in the concrete deck or in the top flange of the steel girder. In such cases, the entire web of the girder is in tension and D_{cp} is to be taken as zero according to Article D6.3.2. When D_{cp} is equal to zero, the web-slenderness requirements in the AASHTO LRFD BDS based on D_{cp} are assumed automatically satisfied. The location of the plastic neutral axis (*PNA*) for composite sections in positive flexure can be determined from the conditions listed in Table D6.1-1 (see the Discussion of Article D6.1 in this Guide). The position of the *PNA* is calculated based on the equilibrium condition that there be no net axial force acting on the assumed fully yielded cross-section.

At sections in negative flexure, D_{cp} is typically larger than the elastic depth of the web in compression, D_c, as plastic strains associated with moments larger than M_y are incurred. The location of the *PNA* for composite sections in negative flexure and for noncomposite sections can be determined from the conditions listed in Table D6.1-2. For noncomposite sections, the terms related to the longitudinal reinforcement in Table D6.1-2 should be set equal to zero. In calculating D_{cp} in regions of negative flexure, the concrete deck is assumed not to be effective in tension. Therefore, in most cases, the plastic neutral axis will be located in the web. For rare cases in which the plastic neutral axis is in the top flange and the entire web is in compression, D_{cp} is to be taken equal to the web depth D. For composite sections in negative flexure where the plastic neutral axis is located in the web, D_{cp} may simply be computed using Eq. D6.3.2-2.

For further information on the depth of the web in compression at the plastic moment, D_{cp}, and example calculations of D_{cp}, consult Section 6.4.5.4.2 of the Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures. The reader is cautioned that the Reference Manual for NHI Course 130081 has not yet been updated to reflect changes made in the 10th Edition of the AASHTO LRFD BDS; this Reference Manual still contains significant amounts of valuable information, but may occasionally present guidance which contradicts the 10th Edition of the AASHTO LRFD BDS.

Simple Span Bridges:

For the routine simple span bridges covered by this Guide, D_{cp} for the composite section at the strength limit state will most always be in the top flange or concrete deck and thus may be taken equal to zero if that is the case. D_{cp} for the noncomposite section may be needed if the section has a nonslender web and the provisions of Article A6.3.3 are optionally used to determine the nominal lateral-torsional buckling resistance for use in checking Eq. 6.10.3.2.1-2 during construction in order to include the beneficial effect of the St. Venant torsional constant, J (see the Discussion of Articles A6.3.3 and 6.10.3.2.1 in this Guide).

Multi-span Continuous Rolled Beam Bridges and Multi-span Continuous Plate Girder Bridges:

For the routine multi-span continuous rolled beam bridges covered by this Guide, D_{cp} for composite sections in regions of positive flexure at the strength limit state will most always be in the top flange or concrete deck and thus may be taken equal to zero if that is the case. D_{cp} must be calculated for these bridges if composite sections or noncomposite sections (if studs are not provided) in regions of negative flexure satisfy the restrictions specified in Article 6.10.6.2.3 (see the Discussion of Article 6.10.6.2.3 in this Guide) and the optional provisions of Appendix A6 are used to determine the nominal flexural resistance at the strength limit state, which is typically the case and is strongly recommended for rolled-beam bridges. In such cases, D_{cp} is used to determine whether the section qualifies as either a compact web or a noncompact web section (see the Discussion of Article A6.2.1 in this Guide). D_{cp} for the noncomposite section may be needed if the section has a nonslender web and the provisions of Article A6.3.3 are optionally used to determine the nominal lateral-torsional buckling resistance for use in checking Eq. 6.10.3.2.1-2 during construction in order to include the beneficial effect of the St. Venant torsional constant, J (see the Discussion of Articles A6.3.3 and 6.10.3.2.1 in this Guide).

D6.4 LATERAL TORSIONAL BUCKLING EQUATIONS FOR C_B > 1.0, WITH EMPHASIS ON UNBRACED LENGTH REQUIREMENTS FOR DEVELOPMENT OF THE MAXIMUM FLEXURAL RESISTANCE

D6.4.1 By the Provisions of Article 6.10.8.2.3

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article should always be invoked when calculating the nominal lateral-torsional buckling resistance of a prismatic unbraced length under uniform bending according to the provisions of Articles 6.10.8.2.3a and 6.10.8.2.3b, and when a moment-gradient modifier, C_b, is calculated for the unbraced length under consideration and is greater than 1.0 (see the Discussion of Articles 6.10.8.2.3a and 6.10.8.2.3b in this Guide). The provisions of this Article can result in the nominal flexural resistance of the compression flange reaching the plateau strength (F_{max}) for lateral-torsional buckling at significantly larger unbraced lengths under moment-gradient conditions when the effects of the moment-gradient are included in determining the limits on the unbraced length, L_b (see the dashed curve in Figure C6.10.8.2.1-1).

Simple Span Bridges:

The provisions of this Article are not applicable for the routine simple span bridges covered by this Guide at the strength limit state because the top flanges are continuously braced. These provisions may be applicable to these bridges when computing the lateral-torsional buckling resistance of a prismatic unbraced length of the discretely braced top (compression) flange of the noncomposite section during construction using the provisions of Articles 6.10.8.2.3a and 6.10.8.2.3b for use in Eq. 6.10.3.2.1-2 (see the Discussion of Article 6.10.3.2.1 in this Guide), which is not recommended for rolled-beam bridges in particular, and when C_b is calculated for the unbraced length under consideration and is greater than 1.0. Note however that C_b is typically taken equal to 1.0 when checking lateral-torsional buckling of the critical noncomposite section in regions of positive flexure during construction; otherwise, use Eq. 6.10.8.2.3b-1 to compute C_b (see the Discussion of Article 6.10.8.2.3b in this Guide).

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

The provisions of this Article are applicable for the routine multi-span continuous rolled-beam and plate girder bridges covered by this Guide at the strength limit state when computing the lateral-torsional buckling resistance of a prismatic unbraced length of the discretely braced bottom (compression) flange in regions of negative flexure using the provisions of Articles 6.10.8.2.3a and 6.10.8.2.3b (which is not recommended for rolled-beam bridges), and when C_b is calculated for the unbraced length under consideration and is greater than 1.0. These provisions are also applicable for these bridges when computing the lateral-torsional buckling resistance of a prismatic unbraced length of the discretely braced compression flanges of the noncomposite section during construction using the provisions of Articles 6.10.8.2.3a and 6.10.8.2.3b for use in Eq. 6.10.3.2.1-2 (which is not recommended for rolled-beam bridges, see the Discussion of Article 6.10.3.2.1 in this Guide), and when C_b is calculated for the unbraced length under consideration and is greater than 1.0. Note however that C_b is typically taken equal to 1.0 when checking lateral-torsional buckling of the critical noncomposite section in regions of positive flexure during construction; otherwise, use Eq. 6.10.8.2.3b-1 to compute C_b (see the Discussion of Article 6.10.8.2.3b in this Guide).

D6.4.2 By the Provisions of Article A6.3.3

Determination of applicability, *All Routine Steel I-girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article should always be invoked when calculating the nominal lateral-torsional buckling resistance of a prismatic unbraced length under uniform bending according to the provisions of Articles A6.3.3.1 and A6.3.3.2 in the optional Appendix A6, and when a moment-gradient modifier, C_b, is calculated for the unbraced length under consideration and is greater than 1.0 (see the Discussion of Articles A6.3.3.1 and A6.3.3.2 in this Guide). The provisions of this Article can result in the nominal flexural resistance of the compression flange reaching the plateau strength (M_{max}) for lateral-torsional buckling at significantly larger unbraced lengths under moment-gradient conditions when the effects of the moment-gradient are included in determining the limits on the unbraced length, L_b (see the dashed curve in Figure C6.10.8.2.1-1).

Simple Span Bridges:

The provisions of this Article are not applicable for the routine simple span bridges covered by this Guide at the strength limit state because the top flanges are continuously braced. These provisions may be applicable to these bridges when computing the lateral-torsional buckling resistance of a prismatic unbraced length of noncomposite sections with nonslender webs during construction for use in Eq. 6.10.3.2.1-2 (see the Discussion of Article 6.10.3.2.1 in this Guide) using the provisions of Articles A6.3.3.1 and A6.3.3.2 to account for the beneficial effect of the St. Venant torsional constant, J, which is recommended for rolled-beam bridges in particular, and when C_b is calculated for the unbraced length under consideration and is greater than 1.0. Note however that C_b is typically taken equal to 1.0 when checking lateral-torsional buckling of the critical noncomposite section in regions of positive flexure during construction; otherwise, use Eq. 6.10.8.2.3b-1 to compute C_b (see the Discussion of Article 6.10.8.2.3b in this Guide).

Multi-span Continuous Rolled Beam Bridges and *Multi-span Continuous Plate Girder Bridges*:

The provisions of this Article are applicable for the routine multi-span continuous rolled-beam and plate girder bridges covered by this Guide at the strength limit state when computing the lateral-torsional buckling resistance of sections in regions of negative flexure with nonslender webs using the provisions of Articles A6.3.3.1 and A6.3.3.2 (which is recommended for rolled-beam bridges), and when C_b is calculated for the unbraced length under consideration and is greater than 1.0. These provisions are also applicable for these bridges when computing the lateral-torsional buckling resistance of noncomposite sections with nonslender webs during construction for use in Eq. 6.10.3.2.1-2 (see the Discussion of Article 6.10.3.2.1 in this Guide) using the provisions of Articles A6.3.3.1 and A6.3.3.2 to account for the beneficial effect of the St. Venant torsional constant, J (which is recommended for rolled-beam bridges), and when C_b is calculated for the unbraced length under consideration and is greater than 1.0. Note however that C_b is typically taken equal to 1.0 when checking lateral-torsional buckling of the critical noncomposite section in regions of positive flexure during construction; otherwise, use Eq. 6.10.8.2.3b-1 to compute C_b (see the Discussion of Article 6.10.8.2.3b in this Guide).

D6.5 CONCENTRATED LOADS APPLIED TO WEBS WITHOUT BEARING STIFFENERS

D6.5.1 General

Determination of applicability, *Simple Span Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

At bearing locations on rolled shapes, and at other locations on built-up sections or rolled shapes subjected to concentrated loads where the loads are not transmitted through a deck or deck system (e.g., a jacking point on a girder or on a diaphragm at an end and/or interior support to facilitate bearing replacement or other maintenance activities), either bearing stiffeners must be provided or else the web must be investigated for the limit states of web local yielding and web crippling (see the Discussions of Articles D6.5.2 and D6.5.3 in this Guide).

The limit state of sidesway web buckling given in the AISC *Specification for Structural Steel Buildings* is not included in the AASHTO LRFD BDS because it governs only for: 1) members subjected to concentrated loads applied directly to the steel section; 2) members for which the compression flange is braced at the load point; 3) members for which the tension flange is unbraced at the load point; and 4) members for which the ratio of D/t_w to L_b/b_{ft} is less than or equal to 1.7. The preceding conditions do not commonly occur in bridge construction.

Simple Span Bridges:

The provisions of this Article are not applicable to the routine simple span plate-girder bridges covered by this Guide at support reactions, at which bearing stiffeners are required (see the Discussion of Article 6.10.11.2 in this Guide). These provisions are applicable to determine if bearing stiffeners are required at support reactions in routine simple span rolled-beam bridges. These provisions are also applicable for the routine simple-span plate-girder or rolled-beam bridges covered by this Guide to determine if bearing stiffeners are required at jacking points on the beam/girder or a diaphragm at the end supports (if provided).

Multi-span Continuous Rolled Beam Bridges:

The provisions of this Article are applicable to the routine multi-span continuous rolled beam bridges covered by this Guide to determine if bearing stiffeners are required at support reactions. These provisions are also applicable for these bridges to determine if bearing stiffeners are required at jacking points on the beam or diaphragm at an end and/or interior support (if provided).

Multi-span Continuous Plate Girder Bridges:

The provisions of this Article are not applicable to the routine multi-span continuous plate-girder bridges covered by this Guide at support reactions, at which bearing stiffeners are required (see the Discussion of Article 6.10.11.2 in this Guide). These provisions are applicable for these bridges to determine if bearing stiffeners are required at jacking points on the girder or diaphragm at an end and/or interior support (if provided).

D6.5.2 Web Local Yielding

Determination of applicability, *Simple Span Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

The limit state of web local yielding is intended to prevent localized yielding of the web at the strength limit state due to a high compressive or tensile stress caused by a concentrated load or bearing reaction. These provisions are also used to check for the need for a partial-or full-depth transverse stiffener at the location where the bottom flange becomes horizontal in a variable web-depth member. The routine steel I-girder bridges covered by this Guide are assumed to contain only constant-depth members.

To satisfy this limit state without providing bearing stiffeners, webs subject to compressive or tensile concentrated loads must satisfy Eq. D6.5.2-1. The nominal resistance to the concentrated loading, R_n, in Eq. D6.5.2-1 is determined from Eq. D6.5.2-2 or D6.5.2-3. Eq. D6.5.2-2 applies if the factored bearing reaction, R_u, is an interior-pier reaction or if the factored concentrated load, R_u, is applied at a distance from the end of the member that is greater than d, where d is the depth of the member. Otherwise, Eq. D6.5.2-3 applies.

The concentrated load acting on a rolled shape or built-up section is assumed critical at the toe of the fillet, located a distance k from the outer face of the flange resisting the concentrated load or bearing reaction. For a rolled shape, k is published in the tables in the AISC *Manual of Steel Construction* giving dimensions for the shapes. For a built-up section, k may be taken as the distance from the outer face of the flange to the web toe of the web-to-flange fillet weld.

For an interior concentrated load or interior-pier bearing reaction, the load is assumed to distribute along the web at a slope of 2.5 to 1 and over a distance of $(5k + N)$, where N is the length of bearing. N must be greater than or equal to k at end bearing locations. An interior concentrated load is assumed to be a load applied at a distance from the end of the member greater than d. For an end concentrated load or end reaction, the load is assumed to distribute along the web at the same slope over a distance of $(2.5k + N)$.

Simple Span Bridges:

The provisions of this Article are not applicable to the routine simple span plate-girder bridges covered by this Guide at support reactions, at which bearing stiffeners are required (see the Discussion of Article 6.10.11.2 in this Guide). These provisions are applicable to determine if bearing stiffeners are required at support reactions in routine simple span rolled-beam bridges. These provisions are also applicable for the routine simple-span plate-girder or rolled-beam bridges covered by this Guide to determine if bearing stiffeners are required at jacking points on the beam/girder or a diaphragm at the end supports (if provided).

Multi-span Continuous Rolled Beam Bridges:

The provisions of this Article are applicable to the routine multi-span continuous rolled beam bridges covered by this Guide to determine if bearing stiffeners are required at support reactions. These provisions are also applicable for these bridges to determine if bearing stiffeners are required at jacking points on the beam or diaphragm at an end and/or interior support (if provided).

Multi-span Continuous Plate Girder Bridges:

The provisions of this Article are not applicable to the routine multi-span continuous plate-girder bridges covered by this Guide at support reactions, at which bearing stiffeners are required (see the Discussion of Article 6.10.11.2 in this Guide). These provisions are applicable for these bridges to determine if bearing stiffeners are required at jacking points on the girder or diaphragm at an end and/or interior support (if provided).

D6.5.3 Web Crippling

Determination of applicability, *Simple Span Bridges*: Conditionally applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges*: Applicable.

Determination of applicability, *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

The limit state of web crippling is intended to prevent local instability or crippling of the web due to a high compressive stress caused by a concentrated load or bearing reaction.

To satisfy this limit state without providing bearing stiffeners, webs subject to compressive concentrated loads must satisfy Eq. D6.5.3-1. The nominal resistance to the concentrated loading, R_n, is determined from Eq. D6.5.3-2, D6.5.2-3, or D6.5.2-4. Eq. D6.5.3-2 applies if the factored bearing reaction, R_u, is an interior-pier reaction or if the factored concentrated load, R_u, is applied at a distance from the end of the member that is greater than or equal to $d/2$, where d is the depth of the member. Otherwise, Eq. D6.5.3-3 or D.6.5.3-4 applies depending on the ratio of N/d, where N is the length of bearing. N must be greater than or equal to k at end bearing locations. Eq. D6.5.3-3 applies if the ratio of N/d is less than or equal to 0.2; otherwise, Eq. D6.5.3-4 applies.

Simple Span Bridges:

The provisions of this Article are not applicable to the routine simple span plate-girder bridges covered by this Guide at support reactions, at which bearing stiffeners are required (see the Discussion of Article 6.10.11.2 in this Guide). These provisions are applicable to determine if bearing stiffeners are required at support reactions in routine simple span rolled-beam bridges. These provisions are also applicable for the routine simple-span plate-girder or rolled-beam bridges covered by this Guide to determine if bearing stiffeners are required at jacking points on the beam/girder or a diaphragm at the end supports (if provided).

Multi-span Continuous Rolled Beam Bridges:

The provisions of this Article are applicable to the routine multi-span continuous rolled beam bridges covered by this Guide to determine if bearing stiffeners are required at support reactions. These provisions are also applicable for these bridges to determine if bearing stiffeners are required at jacking points on the beam or diaphragm at an end and/or interior support (if provided).

Multi-span Continuous Plate Girder Bridges:

The provisions of this Article are not applicable to the routine multi-span continuous plate-girder bridges covered by this Guide at support reactions, at which bearing stiffeners are required (see the Discussion of Article 6.10.11.2 in this Guide). These provisions are applicable for these bridges to determine if bearing stiffeners are required at jacking points on the girder or diaphragm at an end and/or interior support (if provided).

D6.6 ELASTIC LTB LOAD RATIO, γ_e, FOR NONPRISMATIC UNBRACED LENGTHS OF I-SECTION MEMBERS

D6.6.1 General

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article are used to estimate the elastic LTB load ratio, γ_e, for nonprismatic unbraced lengths of I-section members. A nonprismatic unbraced length is defined as an unbraced length between cross-frames or diaphragms in which the member cross-section and/or yield strength varies along the length.

This Article is conditionally applicable for routine steel multi-span continuous I-girder bridges, and only partially applicable for routine simple span I-girder bridges covered by this Guide, as described further in the Discussion of Articles 6.10.8.2.1 and A6.3.1 in this Guide, as applicable.

The elastic LTB load ratio, γ_e, is used in the computation of the elastic LTB resistance, F_e, for a nonprismatic unbraced length in the provisions of Article 6.10.8.2.3c (see the Discussion of Article 6.10.8.2.3c in this Guide) for slender-web section members, and in the provisions of Article A6.3.3.3 (see the Discussion of Article A6.3.3.3 in this Guide) for compact-web or noncompact-web section members. See the Commentary for Article 6.10.6.2.3 and the Discussion of Article 6.10.6.2.3 in this Guide for further discussion on the definition and categorization of compact web, noncompact web, and slender web sections. γ_e is a constant by which the calculated design moments and stresses at the governing cross-section would need to be scaled to reach the theoretical elastic LTB load level (refer to Figures C6.10.8.2.3a-1 and CA6.3.3.3-1 in the Commentary of those Articles).

The elastic LTB load ratio, γ_e, may be calculated using one of the three alternative methods specified in Articles D6.6.2, D6.6.3, or D6.6.4 (i.e., Method A, Method B, or Method C, respectively - see the discussion of Articles D6.6.2, D6.6.3, and D6.6.4 in this Guide). There is no particular favor given to any of the alternative methods to calculate γ_e. The designer is free to evaluate each method and choose which one is easier to use, better suited to the situation at hand, etc. The methods should give reasonably comparable results in most cases. The more approximate Methods A and B were determined to be viable and are just different approaches to investigate a very complex problem in a reasonable fashion. The methods do not supersede each other.

For further information and design examples illustrating the application of the LTB provisions for nonprismatic unbraced lengths using Methods A, B, and C to compute γ_e, consult the AASHTO Nonprismatic Girder Design Guide.

D6.6.2 Calculation of the Elastic LTB Load Ratio, γ_e—Method A

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article are used to estimate the elastic LTB load ratio, γ_e, for nonprismatic unbraced lengths of I-section members using optional Method A. See the Discussion of Article D6.6.1 in this Guide for the definition of a nonprismatic unbraced length and the elastic LTB load ratio, γ_e.

This Article is conditionally applicable for routine steel multi-span continuous I-girder bridges, and only partially applicable for routine simple span I-girder bridges covered by this Guide, as described further in the Discussion of Articles 6.10.8.2.1 and A6.3.1 in this Guide, as applicable.

Method A is based on the approach documented in AISC Design Guide 25 to estimate the elastic LTB load ratio, γ_e, for general nonprismatic unbraced lengths of I-section members, including members with a variable web depth and with or without cross-section transitions within the unbraced length under consideration, subject to single- or reverse curvature bending. Alternatively, Method A can also be used to provide for a more refined and potentially accurate solution for prismatic noncomposite unbraced lengths of singly symmetric members subject to reverse curvature bending and for prismatic composite unbraced lengths subject to reverse curvature bending.

Using Method A, γ_e is computed from Eq. D6.6.2-1, which is derived from the relationship given by Eq. CD6.6.2-1 shown in the Commentary for this Article and written at the location within the unbraced length where M_u/M_{e1} is maximum, including the end cross-sections. The term $\gamma_e M_u$ in Eq. CD6.6.2-1 is the moment at this cross-section at incipient elastic buckling of the unbraced length under consideration with M_u calculated from the moment envelope values that produce the largest flexural compression in the flange under consideration, M_{e1} is the elastic buckling moment associated with the direction of M_u at this location, calculated taking $C_b = 1.0$, and taken equal to $F_{e1}S_{xc}$.

F_{e1} is the elastic LTB stress at this location determined from Eq. 6.10.8.2.3b-2 or A6.3.3.2-1, as applicable, with C_b taken equal to 1.0 (see the Discussion of Articles 6.10.8.2.3b and A6.3.3.2 in this Guide), and S_{xc} is the elastic section modulus about the major-axis of the section to the compression flange. In the calculation of F_{e1}, if the web of the member is slender at any cross-section of the unbraced length, Eq. 6.10.8.2.3b-2 is to apply. The radius of gyration for LTB, r_t, may be taken as specified in Eq. 6.10.8.2.3b-3 in the calculation of F_{e1} (see the Discussion of Article 6.10.8.2.3b in this Guide).

C_{be} in Eq, D6.6.2-1 is a moment-gradient modifier for elastic LTB for use in Method A given by Eq. D6.6.2-2, which accounts for the variation in M_u relative to M_{e1} among the points considered in the equation for C_b. In addition to the ratio of $(M_u/M_{e1})_{max}$ as described above, Eq. D6.6.2-2 requires the input of the ratios of the absolute value of the factored major-axis bending moments, M_A, M_B, and M_C at one-quarter, one-half, and three-quarters of the unbraced length under consideration (points A, B, and C), respectively, calculated from the moment envelope values that produce the largest flexural compression in the flange under consideration at these points, or the smallest flexural tension in this flange if the flange is never in compression at the point, to the elastic LTB moment, M_{e1A}, M_{e1B}, and M_{e1C}, at each quarter point, taken equal to $F_{e1}S_{xc}$ (calculated as described above) based on the direction of bending at the section. Since concurrent moments are normally not tracked in the analysis, it is convenient and considered acceptable to utilize the factored worst-case moments from the live load moment envelopes in conjunction with other factored moment diagrams for calculation of C_{be}. For points A, B, or C in unbraced lengths of noncomposite or composite section members where the flange under consideration is subjected to compression and is continuously braced anywhere within either quarter portion of the unbraced length adjacent to the point under consideration, the moment corresponding to that point, A, B, or C, is to be taken equal to zero in Eq. D6.6.2-2. For prismatic section members subjected to reverse curvature bending, only two values of M_{e1} are needed in Eq. D6.6.2-2, one for each sign of the bending moment. The C_{be} value from Eq. D6.6.2-2 may be taken equal to 1.0 as a conservative simplification.

χ in Eq. D6.6.2-1 is a nonprismatic geometry modification factor that modifies C_{be} to account for the stability effects induced by attributes of the nonprismatic geometry that are independent of moment-gradient effects. χ is taken as 1.0 for prismatic unbraced lengths and determined from the provisions of Articles D6.6.2.1 and D6.6.2.2, as applicable, for nonprismatic unbraced lengths (see the Discussion of Articles D6.6.2.1 and D6.6.2.2 in this Guide).

For further information and design examples illustrating the application of the LTB provisions for nonprismatic unbraced lengths using Method A to compute γ_e, consult the AASHTO Nonprismatic Girder Design Guide.

D6.6.2.1 Nonprismatic Geometry Modification Factor, χ, for I-Section Members with Prismatic Flanges and a Linear or a Concave Curved Variation of the Web Depth within the Unbraced Length under Consideration

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article are used to calculate the nonprismatic geometry modification factor, χ, used in Eq. D6.6.2-1 for computing the elastic LTB load ratio, γ_e, using Method A for I-section members with prismatic flanges (i.e., a prismatic over the unbraced length) and a linear or a concave curved variation of the web depth within the unbraced length under consideration. For members with a convex curved variation of the web depth within the unbraced length under consideration, the provisions of this Article may be used by approximating the variation of the web depth as a linear variation of the depth. See the Discussion of Articles D6.6.1 for a definition of the elastic LTB load ratio, γ_e, and the Discussion of Articles D6.6.1 and D6.6.2 for further information on Method A. The routine steel I-girder bridges covered by this Guide are assumed to contain only constant-depth members and so the provisions of this Article are not applicable.

For further information and design examples illustrating the application of the LTB provisions for nonprismatic unbraced lengths using Method A to compute γ_e, consult the AASHTO Nonprismatic Girder Design Guide.

D6.6.2.2 Calculation of γ_e for I-Section Members with Cross-Section Transitions within the Unbraced Length under Consideration

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article are used to calculate the nonprismatic geometry modification factor, χ, used in Eq. D6.6.2-1 for computing the elastic LTB load ratio, γ_e, using Method A for I-section members with cross-section transitions within the unbraced length under consideration (i.e., a nonprismatic unbraced length). See the Discussion of Articles D6.6.1 for a definition of the elastic LTB load ratio, γ_e, and the Discussion of Articles D6.6.1 and D6.6.2 for further information on Method A.

This Article is conditionally applicable for routine steel multi-span continuous I-girder bridges, and only partially applicable for routine simple span I-girder bridges covered by this Guide, as described further in the Discussion of Articles 6.10.8.2.1 and A6.3.1 in this Guide, as applicable.

As indicated by the bulleted items at the beginning of this Article, when there are one or more cross-section transitions within the quarter-lengths adjacent to Points A, B, or C, M_{e1} in the respective ratio used in Eq. D6.6.2-2 to calculate C_{be} is to conservatively be taken as the *minimum* M_{e1} within the adjacent quarter-lengths (see the Discussion of Article D6.6.2 in this Guide for further information on the calculation of C_{be} and M_{e1}). For quarter point A, the adjacent quarter-lengths are the cross-sections within 0 to $0.5L_b$, where L_b is the unbraced length. For quarter point B, the adjacent quarter-lengths are the cross-sections within $0.25L_b$ to $0.75L_b$. For quarter point C, the adjacent quarter-lengths are the cross-sections within $0.5L_b$ to L_b. This modification accounts for the lack of resolution of Eq. D6.6.2-2, i.e., the quarter-point C_{be} equation samples the M_u/M_{e1} values only at a maximum of four points, and only at three points when the maximum value is right at point A, B, or C.

The nonprismatic geometry modification factor, χ, in Eq. D6.6.2-1 accounts for stability effects induced by geometry and cross-section properties independent of the moment gradient. For I-section members containing cross-section transitions within the unbraced length, and with a noncomposite and/or discretely braced top flange, χ is to be computed from Eq. D6.6.2.2-1 (Note: the language regarding the top flange being "noncomposite and/or discretely braced" was unintentionally omitted from the 10th Edition language and will be inserted in the next edition of the AASHTO LRFD BDS). Otherwise, for an unbraced length with a continuously braced top flange, χ is to be taken as 1.0 since the shear center in that case is effectively located at the top flange at all cross-sections (Note: this was also unintentionally omitted from the 10th Edition language and will be inserted in the next edition).

The term d_{Smax} in Eq. D6.6.2.2-1 represents the maximum shift in the shear center of the steel cross-section due to the transition in the cross-section geometry at any position along the unbraced length, or at any combination of transitions that are less than $0.25L_b$ from one another. Eq. CD6.6.2.2-1 in the Commentary for this Article is provided to assist in the determination of d_{Smax}.

Currently, the vertical distance e to the shear center of the steel cross-section determined from Eq. CD6.6.2.2-1 is measured from the center of the compression flange. To more conveniently determine the shift in the shear center at the cross-section transition, it is recommended that $0.5t_{fc}$ be subtracted from the value of e determined from this equation so that the vertical distance is measured from the inside face of the compression flange. This adjustment will be added to this equation in the next edition of the AASHTO LRFD BDS. Also, in the next edition, b_{ft} and t_{ft} will be changed to b_{f1} and t_{f1} and b_{fc} and t_{fc} will be changed to b_{f2} and t_{f2} in the equation, with $0.5t_{f2}$ subtracted from the resulting value of e so that the vertical distance will be measured from the inside face of Flange 2. Measuring the vertical distance from the inside face of Flange 2 rather than the inside face of the compression flange will allow for the more correct determination of the maximum shift in the shear center at the cross-section transition for the case of an unbraced length subject to reverse curvature. Note that if b_{f2} is greater than b_{f1} and $b_{f2}^3t_{f2}$ is greater than $b_{f1}^3t_{f1}$, then the shear center will be located on the web between the centroid of the cross-section and Flange 2. Otherwise, the shear center will be located on the web between the centroid of the cross-section and Flange 1.

If the member has a variable web depth in addition to cross-section transitions, the total χ factor used in Eq. D6.6.2-1 (see the Discussion of Article 6.6.2 in this Guide) is to be taken as the χ factor specified in Article D6.6.2.1 (see the Discussion of Article 6.6.2.1 in this Guide), with I_{yV}/I_{yP} taken as the corresponding maximum ratio within the unbraced length, multiplied by the χ factor determined from the provisions of this Article as described above. The routine steel I-girder bridges covered by this Guide are assumed to contain only constant-depth members and so the preceding provision in Article D6.6.2.2 is not applicable.

For the calculation of γ_e using the provisions of this Article, the transitions in the cross-section are limited to a change in the lateral moment of inertia of the flanges, calculated as specified in Article 6.10.8.2.3c or Article A6.3.3.3 (see the Discussion of Articles 6.10.8.2.3c and A6.3.3.3 in this Guide), as applicable, of no more than a factor of 2.0 at any transition. Adjacent transitions closer than 25 percent of the unbraced length are to be limited to a total change in the lateral moment of inertia of the flanges of no more than a factor of 2.0.

For further information and design examples illustrating the application of the LTB provisions for nonprismatic unbraced lengths using Method A to compute γ_e, consult the AASHTO Nonprismatic Girder Design Guide.

D6.6.3 Calculation of the Elastic LTB Load Ratio, γ_e—Method B

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article are used to estimate the elastic LTB load ratio, γ_e, for nonprismatic unbraced lengths of I-section members using optional Method B. See the Discussion of Article D6.6.1 in this Guide for the definition of a nonprismatic unbraced length and the elastic LTB load ratio, γ_e.

This Article is conditionally applicable for routine steel multi-span continuous I-girder bridges, and only partially applicable for routine simple span I-girder bridges covered by this Guide, as described further in the Discussion of Articles 6.10.8.2.1 and A6.3.1 in this Guide, as applicable.

Method B is based on the use of a weighted-average section approach and traditional moment-gradient expressions to estimate the elastic LTB load ratio, γ_e, for constant-web-depth I-section members with cross-section transitions within the unbraced length and for I-section members with a variable web depth and with or without cross-section transitions within the unbraced length. For the computation of γ_e using Method B, the nonprismatic unbraced length is replaced with a prismatic unbraced length with effective section properties. The effective section properties are computed using a nonlinear length weighted average approach. Effective top and/or bottom flange widths and thicknesses and/or web thicknesses for nonprismatic flanges and/or webs within the unbraced length are computed from Eqs. D6.6.3-4 through D6.6.3-6, as applicable. These equations essentially *treat all sections larger than the smallest section equivalent to the second smallest section within the unbraced length*, which was found to provide reasonably accurate predictions for the LTB resistance of a nonprismatic member with up to three distinct cross-sections within the unbraced length. For prismatic top and/or bottom flanges and/or webs within the unbraced length under consideration, the effective flange width, $b_{f,eff}$, effective flange thickness, $t_{f,eff}$, and/or effective web thickness, $t_{w\ eff}$, are taken as b_f, t_f, and/or t_w, respectively. Otherwise, effective flange widths and/or thicknesses and/or web thicknesses are computed from Eqs. D6.6.3-4 through D6.6.3-6, as applicable.

For constant-web-depth members within the unbraced length under consideration, the effective web depth, D_{eff}, is simply taken as the web depth, D. The 3rd paragraph of the Commentary for this Article describes how to compute an effective web depth, D_{eff}, using Eqs. D6.6.3-1 through D6.6.3-3 when there is a variation in the web depth within the unbraced length under consideration. The routine steel I-girder bridges covered by this Guide are assumed to contain only constant-depth members and so these provisions are not applicable.

Once the effective depth, widths, and thicknesses are computed, the effective radius of gyration for LTB for the prismatic unbraced length with the effective cross-section, $r_{t\ eff}$, is computed from Eq. D6.6.3-7. $r_{t\ eff}$ is then used to compute the elastic LTB stress for the prismatic unbraced length with the effective cross-section, $F_{e\ eff}$, from either Eq. D6.6.3-8 or D6.6.3-9, as applicable. For unbraced lengths in which the LTB parameters are computed using Article 6.10.8.2.3c, Eq. D6.6.3-8 is used. For unbraced lengths in which the LTB parameters are computed using Article A6.3.3.3, Eq. D6.3.3-9 is used with the St. Venant torsional constant for the prismatic unbraced length with the effective cross-section, J_{eff}, computed using D6.6.3-11.

The moment-gradient modifier for elastic LTB for use in Method B, C_{be}, is given by Eq. D6.6.3-10, and is the same equation used to compute the moment-gradient modifier, C_b, given by Eq. 6.10.8.2.3b-1. The computation and application of the modifier from this equation was described previously in the Discussion of Article 6.10.8.2.3b in this Guide. Note that for the calculation of C_{be} from Eq. D6.6.3-10 for singly symmetric noncomposite members subject to reverse curvature bending, the value of ρ_{Top} in Eq. C6.10.8.2.3b-3 for the factor R_m should be taken as the value of ρ_{Top} for the smallest section, i.e., the section with the smallest section modulus, within the unbraced length under consideration. <u>Alternatively, the value of ρ_{Top} for the effective cross-section may be</u>

used to calculate the R_m factor as the difference in the resulting factor is relatively insignificant (see the Discussion of Article 6.10.8.2.3b in this Guide for more information on the C_b and R_m factors).

After computing $F_{e\ eff}$, the elastic LTB load ratio, γ_e, is then to be computed from Eq. D6.3.3-12. $M_{u\ max}$ in Eq. D6.6.3-12 is the maximum factored major-axis bending moment within the unbraced length under consideration, including the end cross sections, calculated from the moment envelope values that produces the largest flexural compression in the flange under consideration, and $S_{xc\ eff}$ is the elastic section modulus about the major axis of the section to the compression flange of the effective cross-section.

For the calculation of γ_e using the provisions of this Article, the transitions in the cross-section are limited to a change in the lateral moment of inertia of the flanges, calculated as specified in Article 6.10.8.2.3c or Article A6.3.3.3 (see the Discussion of Articles 6.10.8.2.3c and A6.3.3.3 in this Guide), as applicable, of no more than a factor of 2.0 at any transition. Adjacent transitions closer than 25 percent of the unbraced length are to be limited to a total change in the lateral moment of inertia of the flanges of no more than a factor of 2.0.

For further information and design examples illustrating the application of the LTB provisions for nonprismatic unbraced lengths using Method B to compute γ_e, consult the AASHTO Nonprismatic Girder Design Guide.

D6.6.4 Calculation of the Elastic LTB Load Ratio, γ_e—Method C

Determination of applicability, *Simple Span Bridges*: Partially applicable.

Determination of applicability, *Multi-span Continuous Rolled Beam Bridges* and *Multi-span Continuous Plate Girder Bridges*: Conditionally applicable.

Discussion:

The provisions of this Article are used to estimate the elastic LTB load ratio, γ_e, for nonprismatic unbraced lengths of I-section members using optional Method C. See the Discussion of Article D6.6.1 in this Guide for the definition of a nonprismatic unbraced length and the elastic LTB load ratio, γ_e.

This Article is conditionally applicable for routine steel multi-span continuous I-girder bridges, and only partially applicable for routine simple span I-girder bridges covered by this Guide, as described further in the Discussion of Articles 6.10.8.2.1 and A6.3.1 in this Guide, as applicable.

In Method C, the elastic LTB load ratio, γ_e, is estimated as the smallest, or controlling, eigenvalue from an elastic buckling analysis using a thin-walled open-section member model or an elastic three-dimensional shell-element model that captures the significant effects of the nonprismatic geometry; i.e., web taper, transitions in the cross-section, and lateral and torsional restraint as appropriate based on the physical characteristics of the member. For further guidance on the best practice to be used to adequately model the unbraced length for such analyses, consult the Commentary to this Article and the AASHTO Nonprismatic Girder Design Guide.

Method C may be used for unbraced lengths in which an I-section member is nonprismatic within the unbraced length and does not meet the specified conditions for the application of Method A in Article D6.6.2 (see the Discussion of Article D6.6.2 in this Guide) or Method B in Article D6.6.3 (see the Discussion of Article D6.6.3 in this Guide), or for cases where a more refined estimate of the member resistance is desired.

For further information and design examples illustrating the application of the LTB provisions for nonprismatic unbraced lengths using Method C to compute γ_e, consult the AASHTO Nonprismatic Girder Design Guide.

APPENDIX E6 NOMINAL COMPRESSIVE RESISTANCE OF NONCOMPOSITE MEMBERS CONTAINING LONGITUDINALLY STIFFENED PLATES

E6.1 NOMINAL COMPRESSIVE RESISTANCE

E6.1.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article are used to determine the nominal compressive resistance of a noncomposite I-section or box-section member subject to axial compression that contains one or more longitudinally stiffened plates.

These provisions are not applicable to the routine steel I-girder bridges covered by this Guide as members in these bridges do not contain longitudinally stiffened plates and are subject to flexure only.

E6.1.2 Classification of Longitudinally Stiffened Plate Panels

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article are used to classify longitudinally stiffened plate panels as either nonslender or slender in a noncomposite I-section or box-section member subject to axial compression that contains one or more longitudinally stiffened plates.

These provisions are not applicable to the routine steel I-girder bridges covered by this Guide as members in these bridges do not contain longitudinally stiffened plates and are subject to flexure only.

E6.1.3 Nominal Compressive Resistance and Effective Area of Plates with Equally-spaced Equal-size Longitudinal Stiffeners

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article are used to determine the effective area of plates with equally-spaced equal-size longitudinal stiffeners for calculating the nominal compressive resistance of a noncomposite I-section or box-section member subject to axial compression that contains one or more longitudinally stiffened plates.

These provisions are not applicable to the routine steel I-girder bridges covered by this Guide as members in these bridges do not contain longitudinally stiffened plates and are subject to flexure only.

E6.1.4 Longitudinal Stiffeners

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article are used to design the longitudinal stiffeners in a noncomposite I-section or box-section member subject to axial compression that contains one or more longitudinally stiffened plates.

These provisions are not applicable to the routine steel I-girder bridges covered by this Guide as members in these bridges do not contain longitudinally stiffened plates and are subject to flexure only.

E6.1.5 Transverse Stiffeners

E6.1.5.1 General

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article are used to design transverse stiffeners when they are utilized to enhance the resistance of a longitudinally stiffened plate in a noncomposite I-section or box-section member subject to axial compression that contains one or more longitudinally stiffened plates.

These provisions are not applicable to the routine steel I-girder bridges covered by this Guide as members in these bridges do not contain longitudinally stiffened plates and are subject to flexure only.

E6.1.5.2 Moment of Inertia

Determination of applicability, *All Routine Steel I-girder Bridges*: Not applicable.

Discussion:

The provisions of this Article are used to determine the moment of inertia requirements for transverse stiffeners when they are utilized to enhance the resistance of a longitudinally stiffened plate in a noncomposite I-section and box-section member subject to axial compression that contains one or more longitudinally stiffened plates.

These provisions are not applicable to the routine steel I-girder bridges covered by this Guide as members in these bridges do not contain longitudinally stiffened plates and are subject to flexure only.

CONCLUSION

The reader is reminded that this Guide is not a substitute for the AASHTO LRFD BDS. This Guide is meant to be used in conjunction with the AASHTO LRFD BDS, with the intent of aiding the reader in navigating and applying its provisions. The Guide also references a number of freely available industry practice documents that the reader is encouraged to consult for additional information and guidance.

REVISION AND ERRATA LIST - DECEMBER 2023

The following list represents corrections incorporated in the December 2023 edition of the "Guide to Navigating Routine Steel Bridge Design".

- The Steel Bridge Design Handbook is no longer managed by FHWA. Since the initial publication of the "Guide to Navigating Routine Steel Bridge Design", the NSBA has taken ownership of the handbook and updated it to the 9th Edition AASHTO Bridge Design Specification.
- Updated all hyperlinks and display text for Steel Bridge Design Handbook references throughout.
- Corrected miscellaneous other hyperlinks.

REVISION AND ERRATA LIST – FEBRUARY 2025

The following list represents corrections incorporated in the Februrary 2025 edition of the "Guide to Navigating Routine Steel Bridge Design".

- Various Determinations of Applicability and Discussions were updated to reflect changes to the AASHTO LRFD BDS 10th Edition.
- New Determinations of Applicability and Discussions were added to address the addition of Article 6.7.4.2 and D6.6 to the AASHTO LRFD BDS 10th Edition.
- Citations of various references were updated to reflect new versions or editions of those references.
- Various editorial changes were made throughout the Guide.