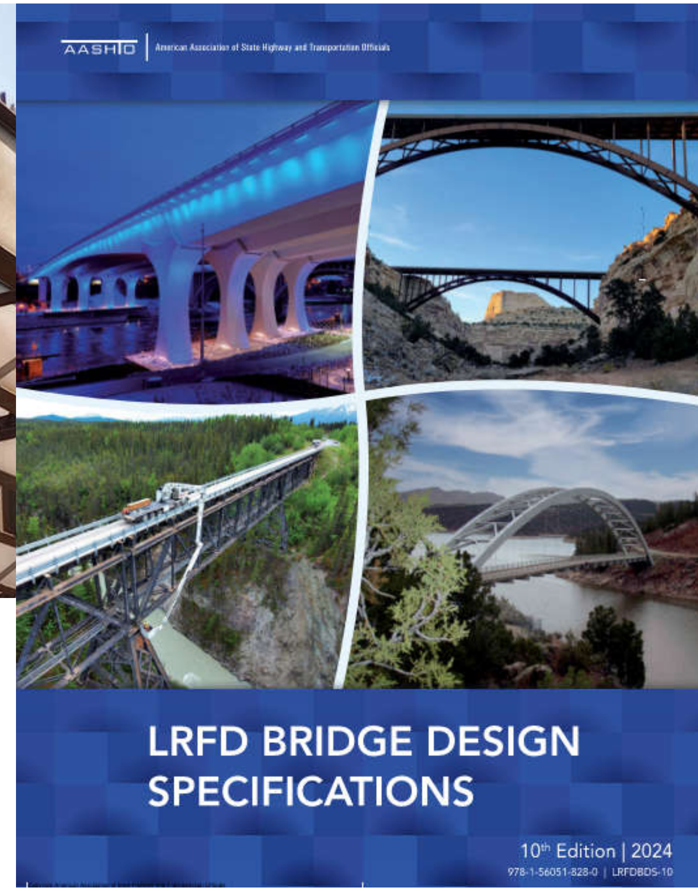


AASHTO 10th Edition Updates

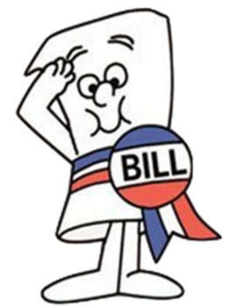
Dusten Olds, PE

04/23/2025

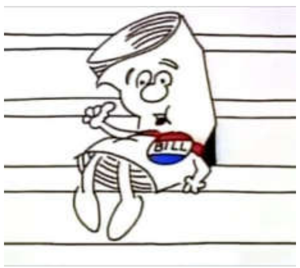
Smarter.
Stronger.
Steel.

AASHTO LRFD BDS Ballot Process

- How a ballot becomes a provision in the AASHTO LRFD specification…
  - Research/change in methods
    - Ballot development

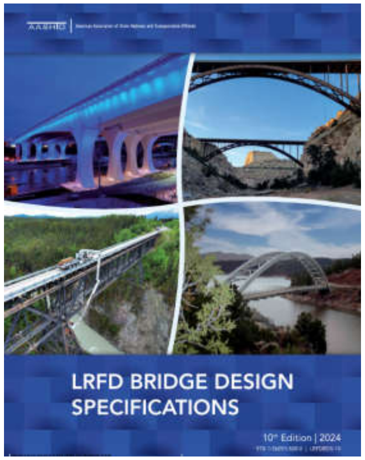


- AASHTO Committee of Bridges and Structures (COBS)
  - Steel and Metals Technical Committee
  - Discussion / changes / adjustments / rework
  - Voting



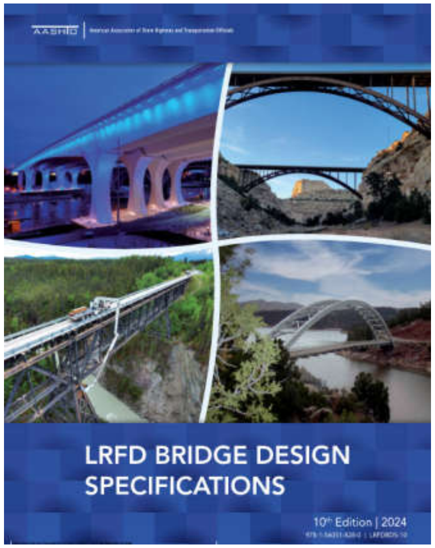
- AASHTO COBS – all 50 States
  - Voting and approval

- AASHTO Publication



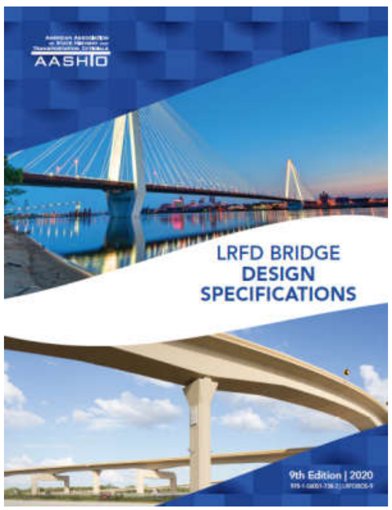
AASHTO LRFD BDS

- For steel I-girder bridges, we have heard:
  - Too big…..
  - Too difficult to follow…..



AASHTO LRFD BDS

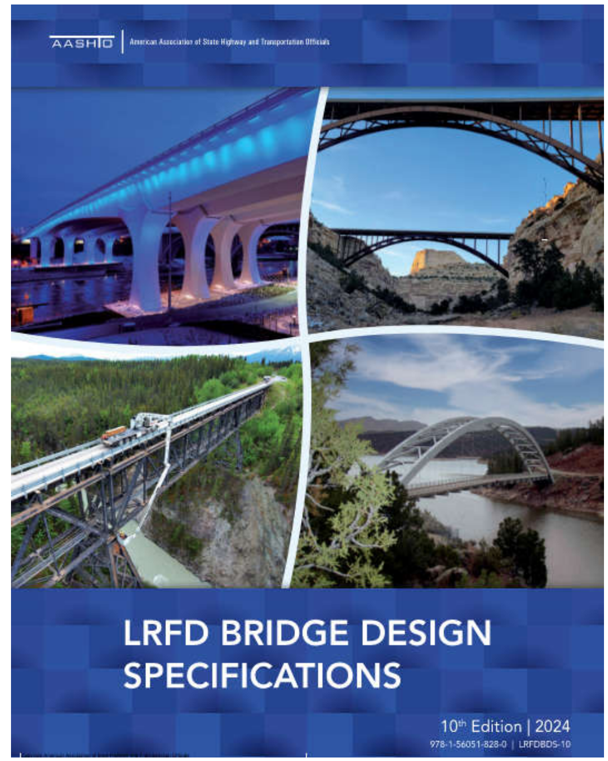
- But, please keep in mind:



Steel Design Revisions in the 10th Edition of the AASHTO LRFD Bridge Design Specifications

- Review some of the more significant updates to Section 6 of the AASHTO LRFD BDS that have occurred since the publication of the 9th Edition in 2020.

- The AASHTO LRFD BDS is now on a 3-year publication schedule:
  - 8th Edition (2017)
  - 9th Edition (2020)
  - 10th Edition (2024)



What You Need to Know: Steel Design Revisions in the 10th Edition of the AASHTO LRFD Bridge Design Specifications

- Outline of the Webinar Presentation:
  - Items Rolled Over from the 2020 to the 2021 COBS Meeting
  - Revisions to the Shear Stud Design Provisions
  - Fatigue of Obliquely Oriented Welded Attachments and Introduction of Half-Round Bearing Stiffeners
  - Modifications to AASHTO Cross-Frame Analysis and Design
  - Lateral Torsional Buckling of Nonprismatic Unbraced Lengths
  - Revisions to Composite Box-Girder Specifications in Article 6.11
  - Slip-Critical vs. Bearing-Type Connections for Bracing Members
  - Shear Resistance of High-Strength Bolts – Threads Included or Excluded
  - Revise ‘FCM’ to ‘NSTM’
  - Other Miscellaneous Revisions in the 10th Edition LRFD BDS
  - Potential Upcoming Revisions for the 11th Edition LRFD BDS (2027)

Recognition

- The general content for this presentation has been previously developed by Mike Grubb, PE.
  - Mike leads the ballot process for many of the updates that occur in the AASHTO LRFD BDS for the steel chapter, and has had a role in this area for at least the last 30 years.
  - Last year at NASCC, Mike received AISC's J. Lloyd Kimbrough Award, which recognizes the pre-eminent steel designers of their era. Mike was just the 13th person to receive the Kimbrough Award since 1941.

M.A. Grubb & Associates, LLC

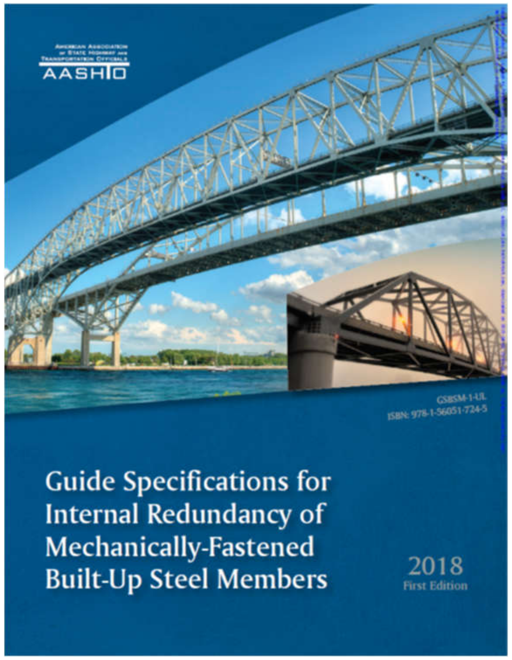
T-14 Ballot Items Rolled Over from the 2020 to the 2021 COBS Meeting

- Revisions to the provisions for determining the flexural resistance of I- or H-shaped members and channels *subject to flexure about their weak axis* in order to bring the provisions up-to-date with the latest provisions given in the 2022 AISC Specification.

- Introduction of a creep reduction factor, K_c, of 0.80 in the determination of the nominal slip resistance of a galvanized faying surface (Class C) or a duplex coated faying surface utilizing a coating producing a higher slip coefficient over a galvanized subsurface.

T-14 Ballot Items Rolled Over from the 2020 to the 2021 COBS Meeting

- Revisions to the AASHTO IRM Guide Specification to incorporate angle-only and two-channel axially loaded tension members, along with some necessary revisions & updates to the design examples.

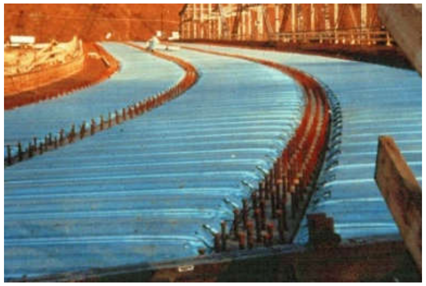


Revisions to Shear Stud Design Provisions (2021)

- Deleted all reference to channel shear connectors.
- Reduced the minimum center-to-center pitch of studs from 6d to 4d.
- Added a pitch correction to account for shear lag across clustered studs.
- Revised the equation for the nominal shear resistance, Q_n, of a stud shear connector at the strength limit state (simpler and somewhat more conservative).

$$\boxed{\cancel{Q_n = 0.5A_{sc}\sqrt{f'_c E_c} \leq A_{sc}F_u}} \Rightarrow Q_n = 0.70A_{sc}F_u$$

- Changed the slope of the fatigue resistance curve for studs in the finite-life region from -3.00 to -5.00. Maintained the constant amplitude threshold, $(\Delta F)_{TH}$, at 7.0 ksi.

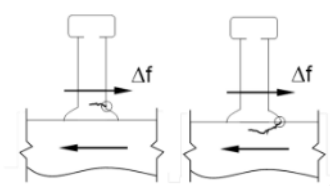


Revisions to Shear Stud Design Provisions – cont'd

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{m}}$$

- Revised the fatigue detail Table 6.6.1.2.3-1 as follows:
  - Changed the exponent in the general equation for the finite-life fatigue resistance from 1/3 to 1/*m*, and added the "growth constant", *m*, to Table 6.6.1.2.3-1 for all fatigue details.
  - Added the fatigue resistance data for studs to Table 6.6.1.2.3-1. Streamlined Article 6.10.10.2.

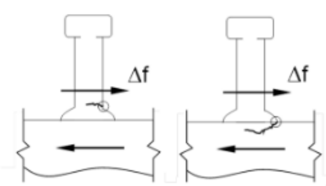
Table 6.6.1.2.3-1 (cont.)—Detail Categories for Load-Induced Fatigue

Description	Category	Constant A (ksi^m)	Fatigue Growth Constant, m	Threshold $(\Delta F)_{TH}$ ksi	75-year $(ADTT)_{SL}$ Equivalent to Infinite Life (trucks per day)	Potential Crack Initiation Point	Illustrative Examples
9.2 Shear connectors or base metal at shear connectors attached by fillet or automatic stud welding (for use in the calculation of Z_r in Eq. 6.10.10.1.2-1 or 6.10.10.1.2-2). Use the horizontal fatigue shear range per unit length, V_{sr}, and Eq. 6.10.10.1.2-1 or 6.10.10.1.2-2, as applicable, to determine the pitch of the shear connectors for fatigue.	N/A	1040 × 10^8	5	7	11,320	Toe of weld growing through the shear connector, or into the base metal	

Revisions to Shear Stud Design Provisions – cont'd

- Revised the fatigue detail Table 6.6.1.2.3-1 as follows:
  - Added the values of the 75-year $(ADTT)_{SL}$ equivalent to infinite life for each detail to Table 6.6.1.2.3-1, and eliminated Tables 6.6.1.2.3-2, 6.6.1.2.5-1, and 6.6.1.2.5-3.
  - Changed Table 6.6.1.2.3-1 from portrait to landscape.

Table 6.6.1.2.3-1 (cont.)—Detail Categories for Load-Induced Fatigue

Description	Category	Constant A (ksi^m)	Fatigue Growth Constant, m	Threshold $(\Delta F)_{TH}$ ksi	75-year $(ADTT)_{SL}$ Equivalent to Infinite Life (trucks per day)	Potential Crack Initiation Point	Illustrative Examples
9.2 Shear connectors or base metal at shear connectors attached by fillet or automatic stud welding (for use in the calculation of Z_r in Eq. 6.10.10.1.2-1 or 6.10.10.1.2-2). Use the horizontal fatigue shear range per unit length, V_{sr}, and Eq. 6.10.10.1.2-1 or 6.10.10.1.2-2, as applicable, to determine the pitch of the shear connectors for fatigue.	N/A	1040×10^8	5	7	11,320	Toe of weld growing through the shear connector, or into the base metal	

Fatigue of Obliquely Oriented Welded Attachments & Introduction of Half-Round Bearing Stiffeners (2021)

- Fatigue characterization of obliquely oriented welded attachments
  - Research at Purdue University
  - New Condition 7.3: fatigue categories transitioning between C' and E as a function of the skew angle, θ (for attachments longer than 4 inches and less than 1-inch thick attached by groove or fillet welds)

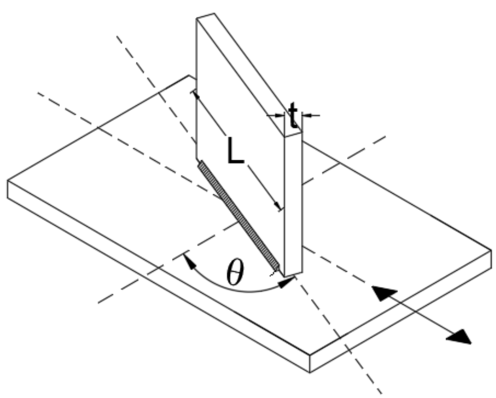


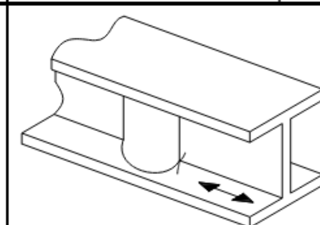
Table 6.6.1.2.3-1 (cont.)—Detail Categories for Load-Induced Fatigue

Description	Category	Constant A (ksi^m)	Fatigue Growth Constant, m	Threshold $(\Delta F)_{TH}$ ksi	75-year $(ADTT)_{SL}$ Equivalent to Infinite Life (trucks per day)	Potential Crack Initiation Point
7.3 Base metal in a longitudinally loaded component at an obliquely oriented detail with a length $L > 4.0$ in. and a thickness t less than 1.0 in. attached by groove or fillet welds (Connor and Korkmaz, 2020):						In the primary member at the weld toe
$\theta \leq 20°$	C'	44 x 10^8	3	12	975	
$20° < \theta \leq 30°$	C	44 x 10^8	3	10	1680	
$30° < \theta \leq 45°$	D	22 x 10^8	3	7	2450	
$45° < \theta < 90°$	E	11 x 10^8	3	4.5	4615	

Fatigue of Obliquely Oriented Welded Attachments & Introduction of Half-Round Bearing Stiffeners – cont'd

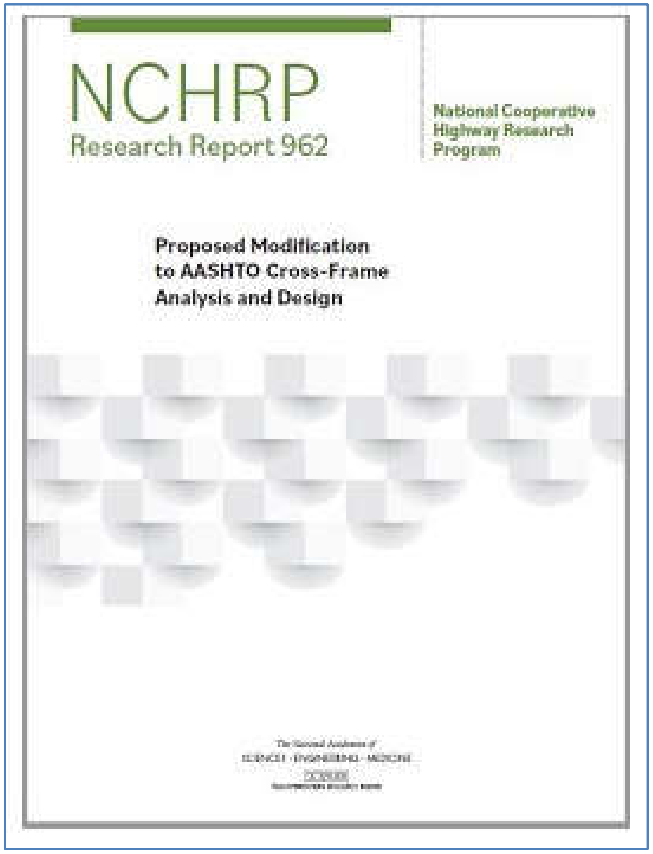
- Research from University of Texas at Austin
- Introduction & fatigue characterization of half-round bearing stiffeners (New Condition 4.2: Category C')

Table 6.6.1.2.3-1 (cont.)—Detail Categories for Load-Induced Fatigue

Description	Category	Constant A (ksi^m)	Fatigue Growth Constant, m	Threshold $(\Delta F)_{TH}$ ksi	75-year $(ADTT)_{SL}$ Equivalent to Infinite Life (trucks per day)	Potential Crack Initiation Point	Illustrative Examples
4.2 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 fillet welds (Quadrato et al., 2010). Fillet welds should be continuous to seal the interior of the half-round.	C'	44×10^8	3	12	975	Initiating from the geometrical discontinuity at the toe of the fillet weld extending into the base metal	

Revisions from NCHRP Project 12-113 (2021)

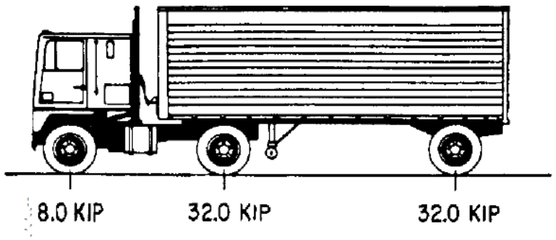
Modifications to AASHTO Cross-Frame Analysis & Design



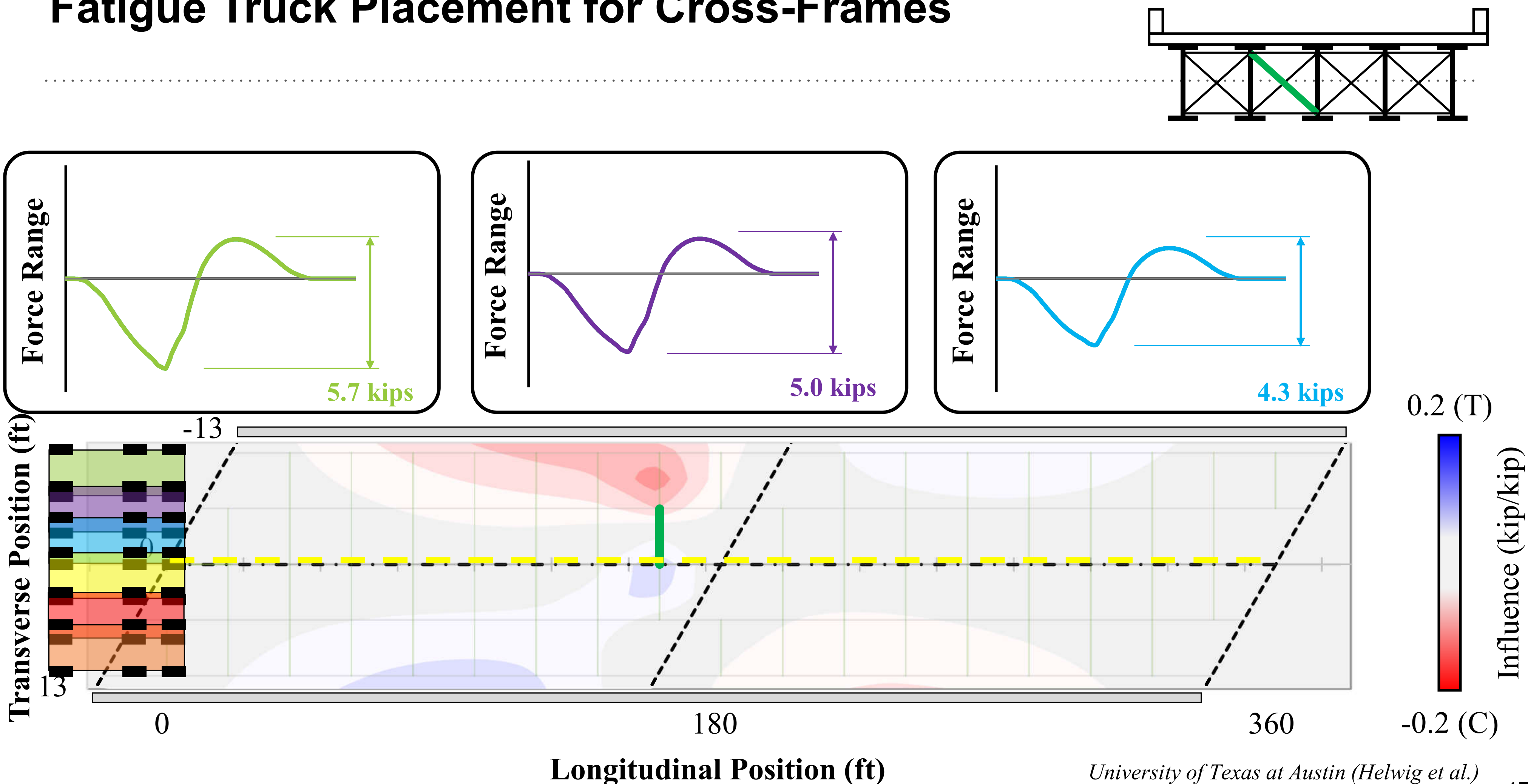
Revisions from NCHRP Project 12-113 - cont'd

Modifications to AASHTO Cross-Frame Analysis & Design

- Revisions to improve the prediction of fatigue force ranges in cross-frame members
  - Specific fatigue truck loading requirements for refined 2D or 3D analyses to better predict the fatigue force ranges in cross-frame members (Article 6.6.1.2.2).
  - Fatigue loading for cross-frames is not the same as for girders
  - Fatigue truck positioned to determine the maximum range of stress or torque, as applicable, with the truck confined to ***one critical transverse position per each longitudinal position*** throughout the length of the bridge in the analysis.



Fatigue Truck Placement for Cross-Frames



University of Texas at Austin (Helwig et al.)

Revisions from NCHRP Project 12-113 - cont’d

Modifications to AASHTO Cross-Frame Analysis & Design

– Multiply Fatigue I and Fatigue II load factors by 0.65 for cross-frames.

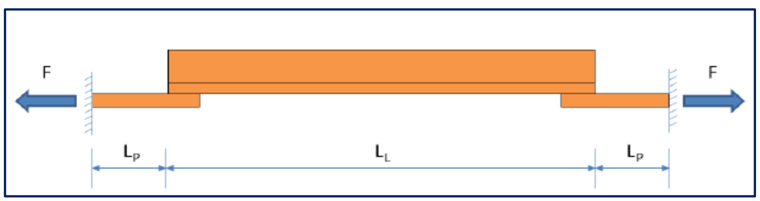
3.4.5—Load Factors for Cross-Frames and Diaphragms at the Fatigue Limit State

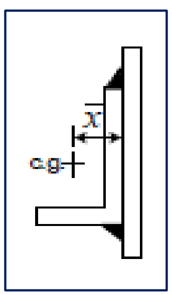
The Fatigue I and II live load factors, γ_{LL}, shall be multiplied by an additional factor of 0.65 when evaluating load-induced fatigue in cross-frames and diaphragms.

Revisions from NCHRP Project 12-113 - cont'd

Modifications to AASHTO Cross-Frame Analysis & Design

– Revisions to the *R* factor in Section 4 of the AASHTO LRFD BDS to better reflect the flexibility of angle and tee-section cross-frame member end connections for composite conditions *in the analysis* (Article 4.6.3.3.4c).





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

In lieu of a more refined analysis, the equivalent axial rigidity of single-angle and tee-section cross-frame members should be taken as 0.65*AE* in the analysis model for the noncomposite condition during construction. In lieu of a more refined analysis, the equivalent axial rigidity of single-angle and tee-section cross-frame members should be taken as 0.75*AE* in the analysis model for the composite condition.

Revisions from NCHRP Project 12-113 - cont'd

Modifications to AASHTO Cross-Frame Analysis & Design

- Recommendations to improve the prediction of cross-frame forces in 2D grid models, and the prediction in general of cross-frame forces in heavily skewed and/or curved bridges.
  - Article 6.7.4.1
  - Article 4.6.3.3.2

Revisions from NCHRP Project 12-113 - cont'd

Modifications to AASHTO Cross-Frame Analysis & Design

- Addition of minimum stability bracing strength and stiffness requirements for cross-frame and diaphragm members in I-girder bridges during the deck placement (similar to the requirements in AISC Appendix Article 6.3.2).
  - AASHTO 6.7.4.2.2

$$\left(\beta_T\right)_{act} \geq \left(\beta_T\right)_{req}$$

$(\beta_T)_{req}$= required stiffness of the torsional brace system (kip-in./rad) calculated as follows:

- For diaphragms and cross-frames, whose depth is at least 0.8 times the beam or girder depth, attached to full-depth connection plates positively attached to both flanges:

$$= \frac{2.4L}{\phi_{sb} nEI_{yeff}} \left(\frac{M_u}{C_b}\right)^2 \qquad (6.7.4.2.2\text{-}2)$$

- Otherwise:

$$= \frac{3.6L}{\phi_{sb} nEI_{yeff}} \left(\frac{M_u}{C_b}\right)^2 \qquad (6.7.4.2.2\text{-}3)$$

Revisions from NCHRP Project 12-113 - cont'd
Modifications to AASHTO Cross-Frame Analysis & Design

– In lieu of a more refined analysis, the actual overall stiffness of the torsional bracing system is to be calculated as follows:

$$(\beta_T)_{act} = \frac{1}{\left(\frac{1}{\beta_{br}} + \frac{1}{\beta_{sec}} + \frac{1}{\beta_g}\right)} \qquad \text{(Eq. 6.7.4.2.2-6)}$$

β_{br} = brace stiffness of the diaphragm or cross-frame that restrains twisting of the beam or girder (kip-in./radian)

β_{sec} = cross-sectional distortion stiffness for stability bracing (kip-in./radian)

β_g = effective in-plane girder stiffness for stability bracing (kip-in./radian)

$$(\beta_T)_{act} \geq (\beta_T)_{req}$$

Revisions from NCHRP Project 12-113 - cont'd
Modifications to AASHTO Cross-Frame Analysis & Design

— In lieu of a more refined analysis, diaphragms or cross-frames in straight rolled-beam or plate-girder bridges with or without skew, and in horizontally curved rolled-beam or plate-girder bridges satisfying all the conditions specified in *AASHTO LRFD* Article 4.6.1.2.4b for neglecting the effects of curvature, are to also satisfy the following *stability bracing strength requirement* for the deck placement sequence as follows:

- For diaphragms and cross-frames, whose depth is at least 0.8 times the beam or girder depth, attached to full-depth connection plates positively attached to both flanges:

$$M_{br} = \frac{2.4L}{nEI_{yeff}}\left(\frac{M_u}{C_b}\right)^2\left(\frac{L_b}{500h_o}\right) \qquad \text{(Eq. 6.7.4.2.2-14)}$$

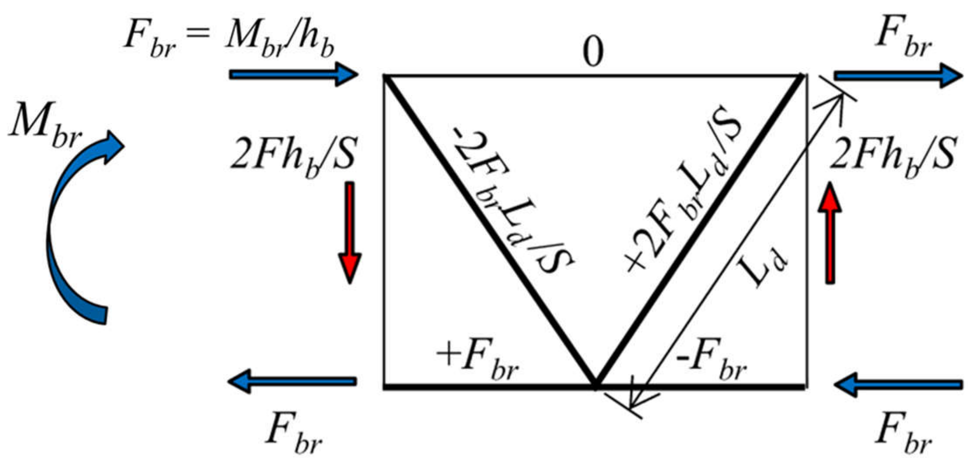
- Otherwise:

$$M_{br} = \frac{3.6L}{nEI_{yeff}}\left(\frac{M_u}{C_b}\right)^2\left(\frac{L_b}{500h_o}\right) \qquad \text{(Eq. 6.7.4.2.2-15)}$$

Revisions from NCHRP Project 12-113 - cont'd
Modifications to AASHTO Cross-Frame Analysis & Design

$$M_{br} = \frac{2.4L}{nEI_{yeff}}\left(\frac{M_u}{C_b}\right)^2\left(\frac{L_b}{500h_o}\right) \quad \text{(Eq. 6.7.4.2.2-14)}$$

$$M_{br} = \frac{3.6L}{nEI_{yeff}}\left(\frac{M_u}{C_b}\right)^2\left(\frac{L_b}{500h_o}\right) \quad \text{(Eq. 6.7.4.2.2-15)}$$



Lateral Torsional Buckling of Nonprismatic Unbraced Lengths (2022)

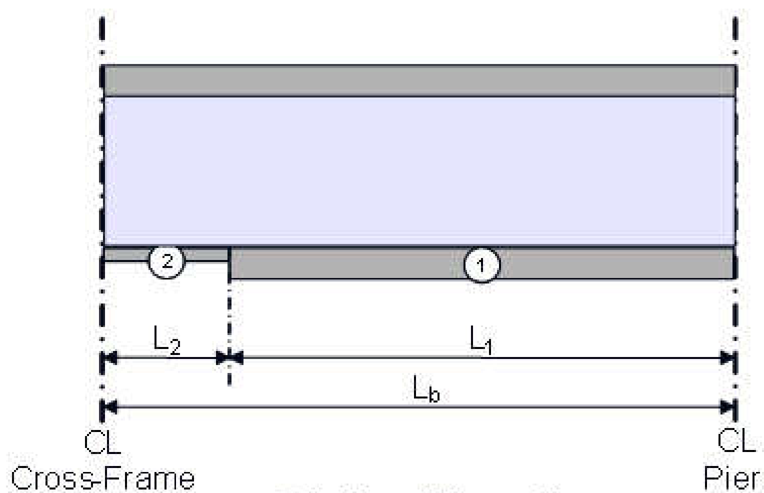
- Agreement AS 20-0026 between AASHTO and Modjeski and Masters, Inc.:

  Flexural Capacity of Steel I-Girders over Interior Piers

  Georgia Tech: Don White, Ryan Slein, Ryan Sherman, others…

  University of Texas at Austin: Todd Helwig, Matt Reichenbach, Mike Engelhardt, others…

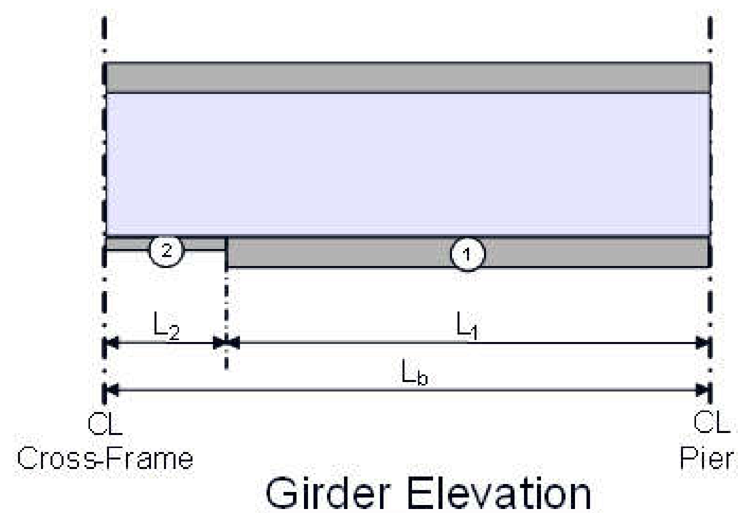
  Lehigh University: Richard Sause, Ian Hodgson



Girder Elevation

Lateral Torsional Buckling of Nonprismatic Unbraced Lengths – cont'd

- *Goals:*
  - Replace the approximate approach with more accurate and robust alternatives for determining the structural capacity of steel I-girders in negative moment regions over interior piers with nonprismatic unbraced lengths, including variable web-depth members.
  - Allow for a more accurate computation of the elastic lateral-torsional buckling resistance of longer nonprismatic unbraced lengths of noncomposite I-section members during temporary construction conditions.



Lateral Torsional Buckling of Nonprismatic Unbraced Lengths – cont'd

Article D6.6 (Appendix D6 - 10th Edition) – Elastic Lateral-Torsional Buckling Load Ratio, γ_e, for Nonprismatic Unbraced Lengths of I-Section Members – Methods A, B, and C

- **METHOD A** (Article D6.6.2)

Based generally on procedures in AISC Design Guide 25 (2nd Edition) with some modifications. Can also be used as an alternative for investigating reverse-curvature bending in a more refined manner in certain cases. Can be used for constant and variable web depths.

- **METHOD B** (Article D6.6.3)

Based on the use of a weighted-average section approach; i.e., using a prismatic unbraced length with effective section properties to "replace" the nonprismatic unbraced length. Can be used for constant and variable web depths.

Lateral Torsional Buckling of Nonprismatic Unbraced Lengths – cont'd

Article D6.6 (Appendix D6 - 10th Edition) – Elastic Lateral-Torsional Buckling Load Ratio, γ_e, for Nonprismatic Unbraced Lengths of I-Section Members – Methods A, B, and C

- **METHOD C** (Article D6.6.4)

Refined analysis – estimate γ_e as the 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 (e.g., SABRE2 or ABAQUS). Use where Method A or B are not applicable or to get a more refined (and likely less conservative) estimate of the member resistance.

Lateral Torsional Buckling of Nonprismatic Unbraced Lengths – cont'd

Other significant revisions:

— Replacement of the current equation for the moment-gradient modifier, C_b, with the quarter-point equation given in the AISC Specification:

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C}$$

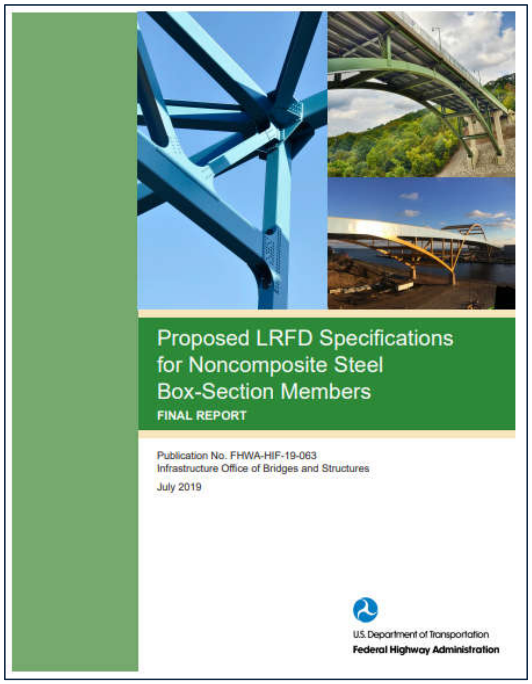
Methods to handle reverse curvature are discussed in the Commentary.

— Replacement of the current equation for the compact unbraced length limit, L_p, with the equation given in the AISC Specification for general I-section members (in both Articles 6.10.8.2.3 and A6.3.3):

$$L_p = 1.1r_t\sqrt{\frac{E}{F_{yc}}}$$

Revisions to Composite Box-Girder Specifications– Article 6.11 (2022)

- Implemented advancements from the FHWA noncomposite box section research, and NCHRP 20-07 Task 415 research on bottom flange proportioning limits, as applicable, into the design provisions for composite steel box sections subject to flexure.

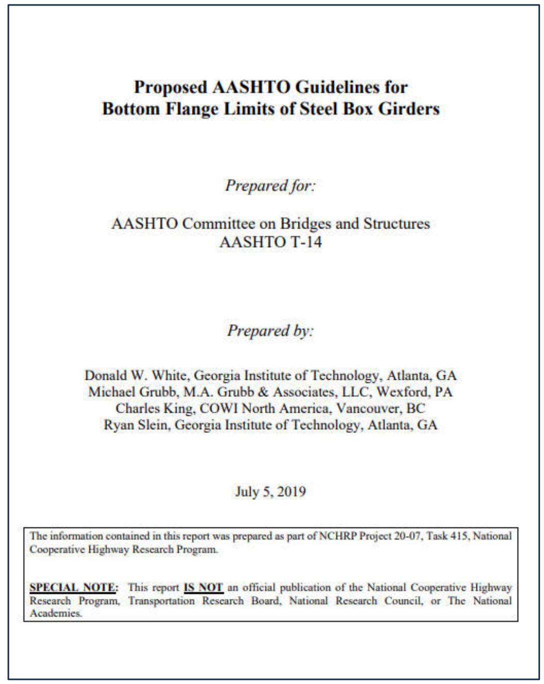


Proposed LRFD Specifications for Noncomposite Steel Box-Section Members

FINAL REPORT

Publication No. FHWA-HIF-19-063
Infrastructure Office of Bridges and Structures
July 2019

U.S. Department of Transportation
Federal Highway Administration



Proposed AASHTO Guidelines for Bottom Flange Limits of Steel Box Girders

Prepared for:

AASHTO Committee on Bridges and Structures
AASHTO T-14

Prepared by:

Donald W. White, Georgia Institute of Technology, Atlanta, GA
Michael Grubb, M.A. Grubb & Associates, LLC, Wexford, PA
Charles King, COWI North America, Vancouver, BC
Ryan Slein, Georgia Institute of Technology, Atlanta, GA

July 5, 2019

The information contained in this report was prepared as part of NCHRP Project 20-07, Task 415, National Cooperative Highway Research Program.

SPECIAL NOTE: This report IS NOT an official publication of the National Cooperative Highway Research Program, Transportation Research Board, National Research Council, or The National Academies.

NATIONAL STEEL BRIDGE ALLIANCE NSBA FOUNDED 1995

Revisions to Composite Box-Girder Specifications– Article 6.11 - cont'd

- Benefits:
  - Greater consistency between the design of composite bridge box girders and the other types of bridge members and components by the AASHTO LRFD provisions.
  - New bottom flange b/t limits that place practical bounds on the use of bottom flanges with extremely large slenderness (particularly in tension) that can result in difficulties during fabrication, construction, and service.
  - Revised bottom-flange compressive resistance equations (that account for post-buckling resistance) that will allow for use of thinner unstiffened flanges where the previous conservative elastic buckling resistances required larger thicknesses or longitudinal stiffening for design.

Revisions to Composite Box-Girder Specifications– Article 6.11 - cont'd

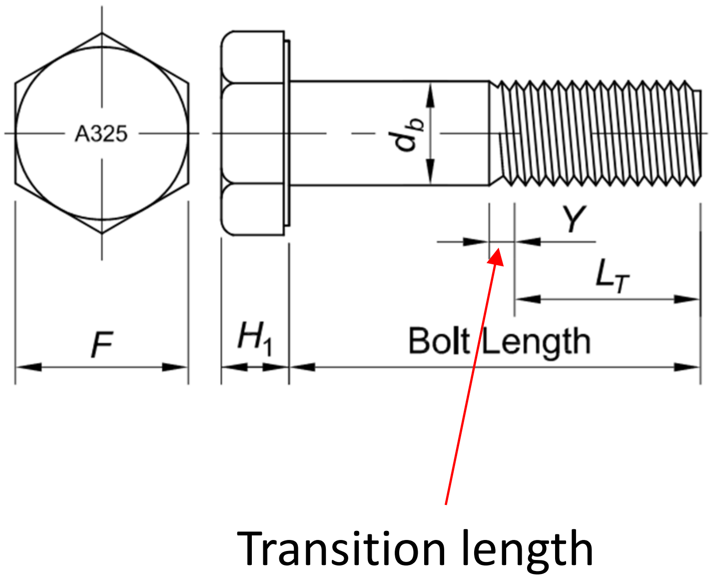
- Benefits:
  - A new constructibility and service plate-buckling requirement that will place additional restrictions on the use of thinner bottom flanges to avoid potential difficulties during construction or in service.
  - New provisions for longitudinally stiffened bottom flanges that will lead to additional economies and eliminate the dramatic increase in the longitudinal stiffener moment of inertia required in the current provisions when the number of stiffeners exceeds one and transverse stiffening is not provided.
  - New primary and secondary member designations for tub-girder bracing members in Table 6.6.2.1-1.

Slip-Critical vs. Bearing-Type Connections for Bracing Members (2022)

- Joints of diaphragm, cross-frame, and lateral bracing members in beam or girder bridges with pretensioned high-strength bolts *installed in standard holes* should be designed only as bearing-type connections (Article 6.13.2.1.1).

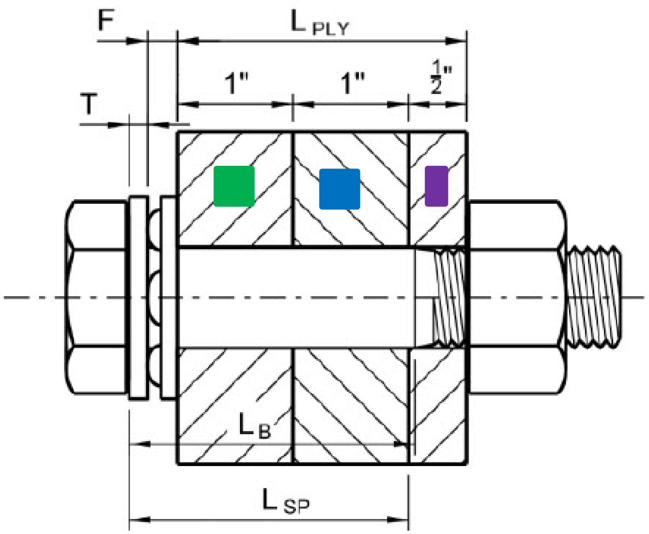
- Field experience has indicated that slip in these connections is not likely and that any slip that may occur in these connections is not anticipated to be detrimental to the geometry or serviceability of the structure.

Shear Resistance of High-Strength Bolts - Threads Included or Excluded (2022)



- Shear planes located in the transition length of high-strength bolts should be considered shear planes with the threads included (Article 6.13.2.7).
- Guidance provided for determining whether threads are excluded from or included in the shear plane considering the bolt transition length (Article C6.13.2.7).

Example - Threads Included or Excluded (Article 6.13.2.7)



Sketch

Given:

Bolt diameter = 7/8”

Ply thicknesses: L_1 = 1.0”, L_2= 1.0”, L_3 = 0.5”

DTI thickness, F = 0.260”

Washer thickness, T = 5/32”

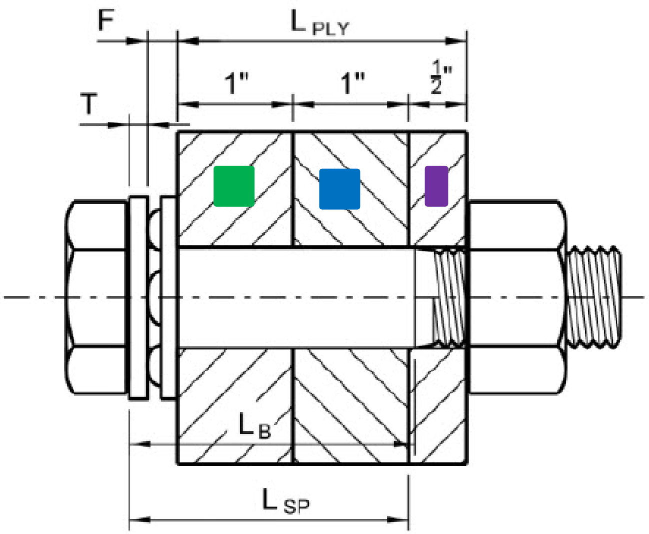
1) Determine if bolt is fully threaded:

Reference RCSC Table 2.5, for 7/8” diam.

Table 2.5
Bolt Lengths Required to Be Fully Threaded in Accordance with ASME B18.2.6

Nominal Bolt Diameter, d_b, in.	Bolt Length, L, in.
½	1¼
⅝	$L \leq$ 1½
¾	$L \leq$ 1¾
⅞	$L \leq$ 2
1	$L \leq$ 2¼
1⅛ & 1¼	$L \leq$ 2¾
1⅜ & 1½	$L \leq$ 3¼

Example - Threads Included or Excluded (Article 6.13.2.7)



Sketch

Given:

Bolt diameter = 7/8”

Ply thicknesses: L_1 = 1.0”, L_2= 1.0”, L_3 = 0.5”

DTI thickness, F = 0.260”

Washer thickness, T = 5/32”

1) Determine if bolt is fully threaded (cont’d):

L_{PLY} = 1.0” + 1.0” + 0.5” = 2.5”

L_{MIN} = L_{PLY} + F + T + 1.125” (RCSC Table C-2.2)

Table C-2.2
Bolt Length Selection

Nominal Bolt Diameter, d_b, in.	To Determine the Required Bolt Length, Add to *Grip + Washer + Direct tension indicator*, in.
½	11/16
⅝	⅞
¾	1
⅞	1⅛
1	1¼

L_{MIN} = 2.5” + 0.260” + 5/32” + 1.125” = 4.041”

→ *RCSC Commentary 2.7 notes to round up to nearest ¼”*

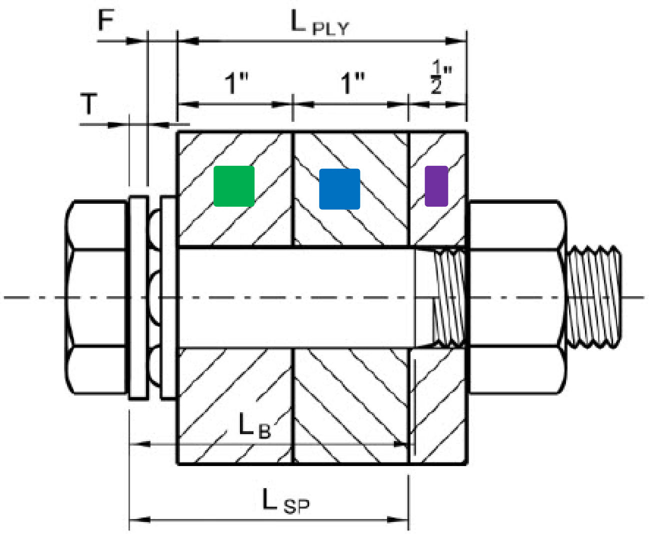
Therefore, minimum nominal bolt length, L_{NOM} = 4.250”

Fully Threaded? (RCSC Table 2.5)

L* = L_{NOM} – F – T = 4.250” – 0.260” – 5/32” = 3.83” > L = 2”

Therefore, bolt is not fully threaded

Example - Threads Included or Excluded (Article 6.13.2.7)



Sketch

Given:

Bolt diameter = 7/8”
Ply thicknesses: L_1 = 1.0”, L_2= 1.0”, L_3 = 0.5”
DTI thickness, F = 0.260”
Washer thickness, T = 5/32”

2) Determine the Minimum Bolt Body Length, L_B:

$L_B = L_{NOM} - L_T - Y$

where: L_T = bolt thread length (RCSC Table C-2.1)
Y = bolt transition thread length (RCSC Table C-2.1)

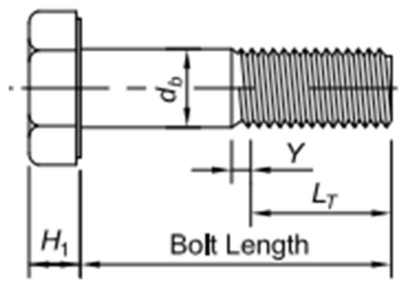


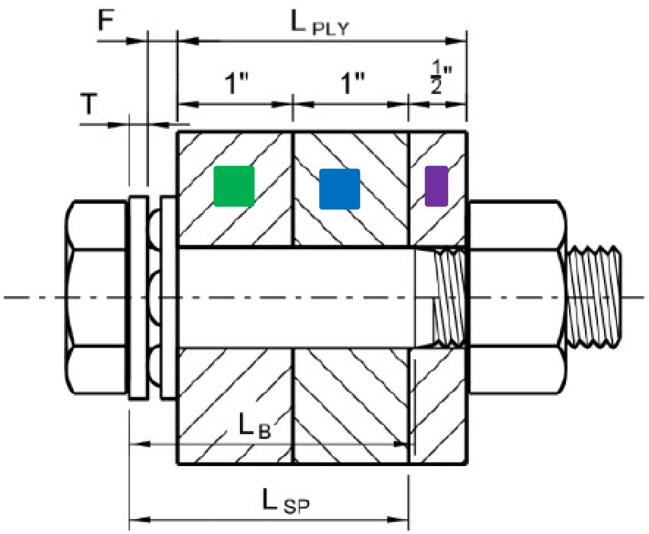
Table C-2.1
Heavy Hex Bolt and Nut Nominal Dimensions

Nomininal Bolt Diameter, d_b	Heavy Hex Bolt Dimensions, in.[a]				Heavy Hex Nut Dims., in.[b]	
	Width across Flats, F	Height, H_1	Thread Length, L_T[c]	Transition Thread Length, Y	Width across Flats, W	Height, H_2
1/2	7/8	5/16	1	3/16	7/8	31/64
5/8	1 1/16	25/64	1 1/4	7/32	1 1/16	39/64
3/4	1 1/4	15/32	1 3/8	1/4	1 1/4	47/64
7/8	1 7/16	35/64	1 1/2	9/32	1 7/16	55/64
1	1 5/8	39/64	1 3/4	5/16	1 5/8	63/64
1 1/8	1 13/16	11/16	2	11/32	1 13/16	1 7/64
1 1/4	2	25/32	2	3/8	2	1 7/32
1 3/8	2 3/16	27/32	2 1/4	7/16	2 3/16	1 11/32
1 1/2	2 3/8	15/16	2 1/4	7/16	2 3/8	1 15/32

[a] See ASME B18.2.6 Table 2.1-1 for additional dimensional information.
[b] See ASME B18.2.6 Table 3.1-1 for additional dimensional information.
[c] See Commentary to Section 2.7 for other thread length configurations.

L_B = 4.250” – 1.5” – 9/32” => **L_B = 2.47”**

Example - Threads Included or Excluded (Article 6.13.2.7)



Sketch

Given:

Bolt diameter = 7/8"

Ply thicknesses: L_1 = 1.0", L_2= 1.0", L_3 = 0.5"

DTI thickness, F = 0.260"

Washer thickness, T = 5/32"

3) Determine the furthest shear plane, L_{SP}:

L_{SP} = length to the location of the furthest shear plane measured from the bolt head

$L_{SP} = T + F + L_{PLY1} + L_{PLY2}$

L_{SP} = 0.260" + 5/32" + 1.0" + 1.0" => **L_{SP} = 2.42"**

4) Compare L_B and L_{SP}:

L_B = 2.47" > L_{SP} = 2.42";

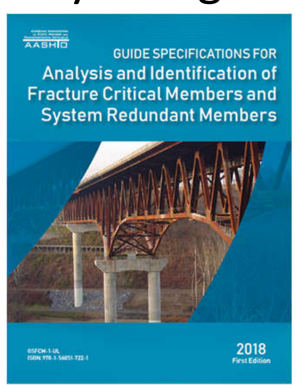
*Therefore, threads (and transition) are **excluded** from the shear planes.*

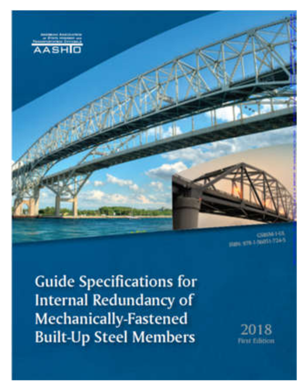
Revise ‘FCM’ to ‘NSTM’ (2023)

- Revise the term ‘Fracture-Critical Member (FCM)’ to ‘Nonredundant Steel Tension Member (NSTM)’ throughout the AASHTO LRFD BDS.
- The National Bridge Inspection Standards (NBIS) were revised in May 2022 and eliminated the term Fracture-Critical Member (FCM) in favor of the term Nonredundant Steel Tension Member (NSTM) because of its implicitly negative connotation and because it was frequently misunderstood by those that did not work regularly with the NBIS.
  - Until such time as other specifications are revised accordingly, for consistency, the terms FCM and NSTM are to be considered synonymous.
  - Also, the new term Load Path Redundant Member (LPRM) is to be considered synonymous with the term nonfracture-critical member.

Revise ‘FCM’ to ‘NSTM’ (2023) – cont’d

- The 2022 NBIS also recognized System Redundant Members (SRMs) and Internally Redundant Members (IRMs) for purposes of alleviating NSTMs from in-service NSTM inspection. The NBIS defines the conditions under which NSTMs may be reclassified as SRMs or IRMs.
- Per a FHWA memo (FHWA Memorandum, May 9, 2022), the AASHTO *Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members* and AASHTO *Guide Specifications for Internal Redundancy of Mechanically-Fastened Built-Up Steel Members*, or an alternative method satisfying either of the two criteria specified in 23 CFR 650.313(f)(1)(i)(B), are considered nationally recognized methods to determine SRMs or IRMs.





Revise ‘FCM’ to ‘NSTM’ (2023) – cont’d

- Fracture Control (FC) Practice is to apply for NSTMs, newly designed SRMs, and primary plate components in newly designed IRMs (Article 6.6.2.2).
- For flexural members, FC practice only applies in the portions of the member located in designated tension zones under Strength Load Combination I where the NSTM, SRM, or IRM classifications apply.
- For materials, FC practice requirements include the more stringent Charpy V-Notch impact energy requirements designated as “F” in ASTM A709/A709M.
- For fabrication, when welding is required, FC Practice requirements include those found in AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* Clause 8.19.8 and Clause 12, the AASHTO *LRFD Steel-Bridge Fabrication Specifications*, and any other Owner-specified requirements.

Revise ‘FCM’ to ‘NSTM’ (2023) – cont’d

- The Engineer need only identify NSTMs, SRMs, and IRMs on the contract plans wherever these classifications apply.
- All other primary members or portions of primary members are LPRMs by default and need not be identified as LPRMs on the contract plans.
- The designation “FC” and identification of the specific FC practice requirements for these members, or portions thereof, is not necessary on the contract plans and should be avoided (Article C6.6.2.2).
  - — Fabricators will use the term “Fracture Control Practice” or the designation “FC” on shop drawings to identify the need for the FC practice requirements.

Revise ‘FCM’ to ‘NSTM’ (2023) – cont’d

- Great summary article discussing this transition…

Implementation of Redundancy Terms under 2022 NBIS

23 U.S.C. 144 (b), Section 650.305 REGARDING REDUNDANCY

Authors: Robert Connor, Heather Gilmer, Jason Lloyd, Ronnie Medlock, and Ed Wasserman.
Editorial Contributions from NSBA Technical Committee – Redundancy Task Force: Ed Wasserman (Chair), Chris Garrell (Secretary), Jamie Farris, Karl Frank, Mike Grubb, Brian Hanks, John Holt, Ronnie Medlock, Duane Miller, and Frank Russo.

Table 1. Requirements for fabrication

Member Classification	Fracture Control Practice Required?	A709 CVN Requirements? (2021)	Identification on Design Drawings?
LPRM	NO	A709 Table 11	NO
NSTM*	YES	A709 Table 12	YES
SRM	YES	A709 Table 12	YES
IRM**	YES	A709 Table 12	YES

*Formerly referred to as FCM
**Primary plate components in newly designed IRMs

https://www.aisc.org/globalassets/nsba/technical-documents/redundancy/b012-23.pdf

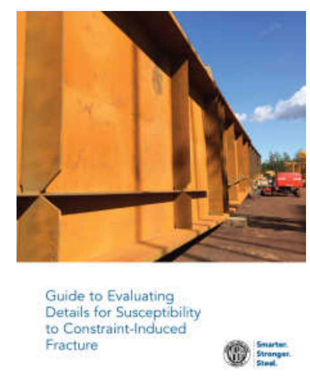
Other Miscellaneous Revisions in the 10th Edition LRFD BDS

- Eq 6.11.2.2-3 shall only apply to built-up tub section members (2021):

$$t_f \geq 1.1 t_w \qquad \text{(Eq. 6.11.2.2-3)}$$

- Revisions to Article 6.8.2.2 and 6.13.5.2 – i.e., further “clean-up” of Table 6.8.2.2-1 containing the shear lag factor, U, for tension members – are made (2021).

- Addition to Article C6.6.1.2.4 summarizing the conditions associated with susceptibility to constraint-induced fracture at welded details along with a brief discussion of intersecting welds, including reference citations for more information (2021)



Potential Upcoming Revisions for the 11th edition LRFD BDS (2027)

- Revised prying force design requirements developed based on streamlining the current AISC design requirements (approved in 2024)
- Reduced design yield resistance for tub/box bolted flange splices governed by compression (approved in 2024)
- Revision to effective in-plane girder stiffness requirement for stability bracing, β_g
- Revised web splice design requirements for bolted splices located in high-moment areas
- Revisions to Article 6.13.3 on welded connection design to align with the 2022 AISC Specification
- Lean-on bracing requirements & other miscellaneous revisions

Questions?

Thank You

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Stronger.
Steel.