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TRANSPORTATION SYMPOSIUM

Florida Steel Design Update

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Transportation Symposium Website

SCAN ME

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~~Outline~~ Topics

- SDG and SDM Updates
- HRBS Updates
- AASHTO 10^{th} Edition
- Specs Updates 460
- Specs Updates 962

DISCLAIMER

- This presentation is based on the draft SDG 2026; changes may occur in the final version.
- It provides updates but is not a replacement for the official document. Always refer to the latest SDG for design work.

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References

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SDG Changes and Updates

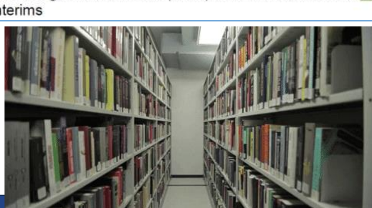
I.6 REFERENCES

A. Except where modified in the ***Structures Manual***, conform to the requirements of the specifications, codes, manuals, and design requirements referenced in this section.

B. AASHTO Publications

1. AASHTO/AWS D1.5M/D1.5- Bridge Welding Code, 2025 9th~~2020 8th~~ Edition.
2. Construction Handbook for Bridge Temporary Works, 2nd Edition (2017)
3. Guide Design Specifications for Bridge Temporary Works, 2nd Edition (2017)
4. The Manual for Bridge Evaluation (MBE), 3rd Edition (2018) with 2020 Interims.
5. LRFD Bridge Design Specifications, 10th~~9th~~ Edition. This document is referenced throughout the Structures Manual as "***LRFD***".
6. LRFD Movable Highway Bridge Design Specifications, 2nd Edition (2007) with 2008, 2010, 2011, 2012, 2014, 2015 and 2018 Interims
7. LRFD Specifications for Structural Supports for Highway Signs, Luminaires an Traffic Signals, 1st Edition (2015) with 2017, 2018, 2019, 2020, and 2022~~2020~~ interims

Updating reference publications dates.



D. Other Publications

1. AISC Steel Construction Manual~~AISC Steel Construction Manual – Fifteenth Edition~~
2. ACI 318 ~~-19(22)~~ Building Code Requirements for Structural Concrete. This document is referenced throughout the Structures Manual as "***ACI CODE-318***".
3. FHWA GEC 11 (FHWA-NHI-10-024 & FHWA-NHI-10-025) Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes - Volumes 1 & 2.
4. AREMA Manual for Railway Engineering
5. Code of Federal Regulations 23 CFR 635.410
6. ASCE Standard ASCE/SEI 7 ~~-10~~ Minimum Design Loads for Buildings and Other Structures
7. ~~2012~~ Florida Accessibility Code
8. Florida Building Code
9. Life Safety Code
10. AWS D1.1/D1.1M: 2025 Structural Welding Code – Steel 25th Edition

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SDG Changes and Updates

5 SUPERSTRUCTURE – STEEL

5.1 GENERAL

A. In addition to ***LRFD*** Section 4, see ***SDG*** 5.13 for straight steel I-girder units and ***SDG*** 5.14 for horizontally curved steel I-girder units.

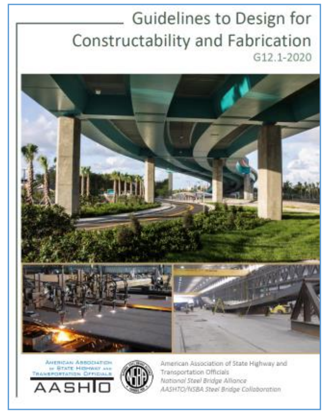
B. Design and detail the cross-frames for steel I-girder superstructures for Steel Dead Load Fit (SDLF). No Load Fit (NLF) and Erected Fit (EF) may be used where appropriate. Total Dead Load Fit (TDLF) is not permitted unless approved by the SSDE, except for phase or widening construction in which the cross-frames located in the closure pour bay must be detailed for TDLF. See ***SDG*** 5.7 and ***SDG*** 7.5.

C. Refer to the following AASHTO/NSBA Steel Collaboration Guidelines (available at http://www.aisc.org/nsba/) with the exceptions as detailed in this chapter:

1. G1.4 - 2006 ***Guidelines for Design and Details***
2. G12.1 - 2020~~2016~~ ***Guidelines to Design for Constructability,***
3. G13.1 - 2019 ***Guidelines for Steel Girder Bridge Analysis***.

D. Only the non-redundant steel superstructure systems listed below are permitted. See ***SDG*** 5.3.2.

1. I-girders in two girder cross sections when approved by the SSDE. See ***SDG*** 10.2 for pedestrian bridges.
2. Truss /arch bridges when approved by the SSDE. See ***SDG*** 10.2 for pedestrian bridges.



Updating AASHTO/NSBA Steel Collaboration Guidelines dates.
Add reference to 10.2 Pedestrian Bridges

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Loads

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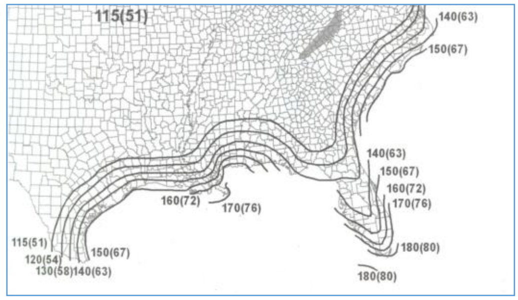
SDG Changes and Updates

Table 2.4.1-1 Design Wind Speed, V

County (Dist)	Design Wind Speed (mph)	County (Dist)	Design Wind Speed (mph)	County (Dist)	Design Wind Speed (mph)
Alachua (2)	130	Hardee (1)	150	Okaloosa (3)	170~~150~~
Baker (2)	130	Hendry (1)	150	Okeechobee (1)	150
Bay (3)	150	Hernando (7)	150	Orange (5)	150
Bradford (2)	130	Highlands (1)	150	Osceola (5)	150
Brevard (5)	170	Hillsborough (7)	150	Palm Beach (4)	170
Broward (4)	170	Holmes (3)	150	Pasco (7)	150
Calhoun (3)	130	Indian River (4)	170	Pinellas (7)	150
Charlotte (1)	170	Jackson (3)	130	Polk (1)	150
Citrus (7)	150	Jefferson (3)	130	Putnam (2)	130
Clay (2)	130	Lafayette (2)	130	St. Johns (2)	150
Collier (1)	170	Lake (5)	150	St. Lucie (4)	170
Columbia (2)	130	Lee (1)	170	Santa Rosa (3)	170~~150~~



Updated design wind speed for Okaloosa and Santa Rosa counties.

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SDG Changes and Updates

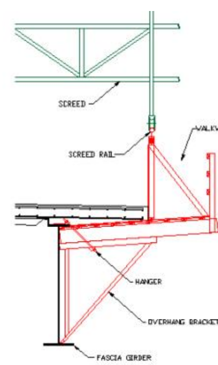
2.13 CONSTRUCTION LOADS

2.13.1 Constructability Limit State Checks

In the absence of more accurate information, the following construction loads can be assumed for investigation of the strength and service limit states during construction in accordance with ***LRFD*** 3.4.2 and ***SDG*** 2.4.3, and for investigation of deck overhang bracket force effects in accordance with ***LRFD*** 6.10.3.4. These loads are applicable to conventional beam or girder superstructures with cast-in-place decks. All construction loads assumed in the design of the structure shall be listed in the plans.

A. Finishing machine load: The finishing machine load shall be per the manufacturer's specifications and be applied as a moving load positioned to produce the maximum response. In the absence of manufacturer's specifications, assume the following loads:

W = Bridge Width (ft)	Total Weight of Finishing Machine (kips)
$26 \le W \le 32$	13~~7~~
$32 < W \le 56$	15~~11~~
$56 < W \le 80$	17~~13~~
$80 < W \le 120$	20~~16~~



Source: NSBA Steel Bridge Design Handbook

Increased assumed finishing machine loads based on updated weight data from consultants/ manufacturers on recent projects

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Terminology Revision (FCM to NSTM)

SDG Changes and Updates

Revise FCM to NSTM

- 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 who did not work regularly with the NBIS.
- In the SDG, SDM, and all our Specs, revised 'Fracture-Critical Member (FCM)' to 'Nonredundant Steel Tension Member (NSTM)'
  - Nonredundant Steel Tension Member (NSTM) = Fracture-Critical Member (FCM)

SDG and SDM Changes and Updates

Update the terminology to adapt NSTM instead of FCM

5.3.2 Fracture (LRFD 6.6.2)

D. Avoid Nonredundant Steel Tension M~~fracture critical m~~embers. Nonredundant Steel Tension~~Fracture critical~~ requirements are expensive due to the intensive welding procedures, base metal and weld tests, and inspections after fabrication.

E. Delete the 2nd paragraph of ***LRFD*** 6.6.2.2 related to System Redundant Members.

Commentary: The identification and use of System Redundant Members are not allowed by **FDOT.**

F. Designate on the plans, all:

1. Primary (main) members. Also, identify areas of primary members that are

7.5.4 Remaining Fatigue Life Estimate for Steel Bridges

A. During the PD&E phase, include an evaluation of the fatigue limit state in the Bridge Analysis if any portion of a steel superstructure unit is retained. Follow the applicable steps outlined below:

Step 1: Determine the $ADTT_{SL}$ per ***LRFD*** 3.6.1.4 using the ADTT information available. The ADTT should be approximately at the mid-point of the assumed 75-year service life of the existing steel superstructure unit.

Step 2: Determine if the Fatigue I (infinite life) or Fatigue II (finite life) limit state applies.

Step 3: If the Fatigue I limit state applies for all members and details, no further evaluation is required. Report this in the Bridge Analysis.

Step 4: If the Fatigue II limit state applies, calculate an approximate remaining fatigue life for each fatigue detail as follows:

B. For steel bridges containing a Nonredundant Steel Tension Member (NSTM)~~Fracture Critical Member (FCM)~~ where the Fatigue II limit state applies, contact the SDO for recommendations.

13.10 PIER DESIGN CONSIDERATIONS - INTEGRAL PIERS

Integral piers present significant design and detailing challenges. Figure 13.10-1 through

16.5 GIRDER ELEVATION

Girder Elevation sheets are required on all bridges with a steel superstructure. The girder elevation may be shown on the Framing Plan, or this may be a separate sheet. Much of the information presented here can be shown in tabular format, as required. Detail girder elevation upstation left to right. At a minimum, include the following on the Girder Elevation sheet:

21.2.5 Main Girder Elevation Drawings

Main Girder Elevation sheets are required for bascule spans. Detail main girder elevation from tail to tip, going left to right. At a minimum, include the following on the Main Girder Elevation sheet:

A. Elevation view of girder. Provide matchlines for girders that require more than 1 sheet.
B. Top/Bottom view of girder (As required). For box main girders, show lateral bracing in top view and show longitudinal stiffeners in bottom view.
C. Shear connector spacing along centerline of girder (centerline of box for box girders).
D. Flange plate sizes.
E. Web plate size.
F. Weld sizes and types. Reference welding symbols at *www.aws.org*.
G. Field splices.
H. Shop splices. Designate optional splices as required.
I. Plates to be Charpy V-Notch (CVN) tested shall be identified. Indicate NSTM~~Fracture Critical Members~~ by notation ~~(FCM)~~.

11

CrossFrames and Diaphragms

Removed what is now included in LRFD 10th edition

These provisions were incorporated into the 2024 Bulletin SDB 24-01 and later in SDG 2025 in preparation for the AASHTO 10th edition release

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SDG Changes and Updates

2.12 LIMIT STATES

2.12.3 Fatigue (LRFD 3.6.1.4.2)

When designing for the Fatigue II limit state, use the following procedure to calculate the ADTT$_{SL}$ (***LRFD*** 3.6.1.4.2):

D. For the following roadways with a Limited Access functional classification (Interstate, Expressway, and Freeway), the *ADTT$_{SL}$* needs to be equal to or greater than:

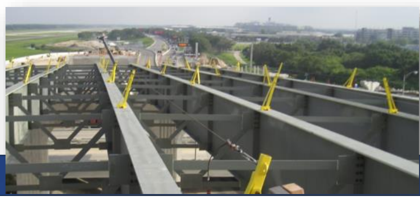
- Ramps (1 lane): 1700,
- Ramps (2 or more lanes): 3100,
- Mainlines and Collector-Distributor (C-D): 3100.

Commentary: It is anticipated that many cross-frames connections will be designed for the Fatigue II Limit State.

~~E. For cross-frames use a value of "n", as defined in ***LRFD*** 6.6.1.2.5, equal to 1.0.~~

Table 6.6.1.2.5-1—Cycles per Truck Passage, *n*

Longitudinal Members	
Simple-Span Girders	1.0
Continuous Girders: 1) near interior support 2) elsewhere	 1.5 1.0
Cantilever Girders	5.0
Orthotropic Deck Plate Connections Subjected to Wheel- Load Cycling	5.0
Trusses	1.0
Transverse Members	
Cross-frames and diaphragms	1.0
Floorbeams Spacing > 20.0 ft	 1.0
Floorbeams Spacing ≤ 20.0 ft	 2.0



Cycle per truck passage, n = 1.0 for Crossframes.
Removed language that is now included in LRFD 10th edition.

TRANSPORTATION SYMPOSIUM

SDG Changes and Updates

5.3.2 Fracture (LRFD 6.6.2)

A. Replace rows 10 and 11 in ***LRFD*** Table 6.6.2.1-1 as shown below. Members in row 11 are exempt from Charpy V-notch testing. See ***SDM*** 2.14 for the definition of skew angle.

Diaphragm and cross-frame members and mechanically fastened or welded cross-frame gusset plates in straight I-girder bridges, skewed straight I-girder units that are designated as Case 1, 2, or 3 (see ***SDG*** 5.13.1), and in horizontally curved I-girder bridges satisfying all the conditions specified in Article 4.6.1.2.4b (for neglecting the effects of curvature)	Secondary
Diaphragm and cross-frame members and mechanically fastened or welded cross-frame gusset plates in straight I-girder units that are designated as Case 4 or 5 (see ***SDG*** 5.13.1), and in horizontally curved I-girder bridges not satisfying one or more of the conditions specified in Article 4.6.1.2.4b (for neglecting the effects of curvature)	Primary

B. Add the following to row 13 of the ***LRFD*** Table 6.6.2.1-1:

Intermediate external diaphragms provided in accordance with ***SDG*** 5.6.3.D are primary members. ~~Add the following rows to ***LRFD*** Table 6.6.2.1-1 immediately after row 11:~~

Table 6.6.2.1-1, (cont.)—Member or Component Designations

For composite box-girder bridges: • Intermediate internal diaphragms that are provided for continuity and their associated intermediate external diaphragms; • External support diaphragms or cross-frame members and mechanically fastened or welded external support cross-frame gusset plates; • Internal support diaphragms in straight bridges with skewed supports or in horizontally curved bridges not satisfying one or more of the conditions specified in Article 4.6.1.2.4c for neglecting the effects of curvature; and • Intermediate external diaphragms provided in bridges with concrete decks designed using the empirical design method to satisfy the design conditions specified in Article 9.7.2.4.	Primary

5.6.3 Cross Frames (LRFD 6.7.4)

A. Design external cross frames as an "X-frame" or a "K-frame" as noted for "I-girders".

B. Design internal cross frames as a "K-frame". Show internal cross frames to be connected by welding or bolting to stiffeners in the fabrication shop.

C. Detail cross frames to be attached to box girders at stiffener locations.

Commentary: An "X-frame" internal diaphragm is easier to fabricate and erect than a "K-frame," but the "K-frame" allows easier inspection access in box girders.

D. Exterior intermediate diaphragms required by ***SDG*** 2.10 shall be plate diaphragms complying with the following:

1. Diaphragms shall be full-width connecting the box girders. Each box girder shall have an interior plate diaphragm colinear with each exterior diaphragm. The

Revised and Removed language now available in the LRFD 10th edition.

SDG Changes and Updates

Note that LRFD only applies the 0.65 factors to CF component forces, and SDG applies it to f_l too

5.13.3 Additional Criteria for Case 1

A. The girder flange lateral bending stresses due to skew effec

B. The forces to design the intermediate and end cross-fram connections are shown in Table 5.13.3-1.

Table 5.13.3-1 Cross-Frame Component Forces

Load Case	Top Chord	Diagor
Intermediate Cross-Frames		
Strength I[1] (kips)	40	60
Fatigue Range[2] (kips)	10	10
Constructibility[3] (kips)	15	10
End Cross-Frames		
Strength I[1,4] (kips)	40	35
Fatigue Range[2] (kips)	10	10
Constructibility[3,4] (kips)	5	5

1. The forces shown for Strength I are already factored.
2. Apply load factors for Fatigue I or II loading combinations as apply a factor of 0.65 (LRFD 3.4.5).
3. The values are for the unfactored force induced into the cros composite weight of the concrete deck and SIP forms. Apply applicable.
4. For end cross-frames directly supporting a free edge of the c do not include effects of the concrete edge beam weight and

Adopted these 0.65 factors for Fat I and II in SDG 2025. Now added references to the LRFD 10th edition.

Table 5.13.4-2 Girder Flange Lateral Bending Stresses Due to Skew Effects

Girder	Location[1]	Unfactored f_L[2] (ksi)	Fatigue Range f_L[3] (ksi)	Cons
Exterior	Near Support at the obtuse corner for Simple Spans	7.5	3.5	
	Near End Supports at the obtuse corner for Continuous Units	7.5 ≤ [2.5+(RDDP/135) *10] ≤ 12	5.5	
	Near Interior Supports of Continuous Units	4.5	N/A[4]	
	Within Span	0	0	
Interior	Near Supports for Simple Spans	7.5	2.5	
	Near End Supports for Continuous Units	10	4.5	
	Near Interior Supports of Continuous Units	7.5	N/A[4]	
	Within Span	0	0	

1. Near is defined as the first two cross-frame lines adjacent to the support. Tra flange lateral bending stress from the value in the table at the first cross-fram a value of zero at the second cross-frame location.
2. Apply a 1.6 or 1.2 load factor for the Strength I and Service II loading combin respectively. These are weighted average load factors (i.e., includes dead ar recommended in the FDOT Research Project BEB13.
3. Apply load factors for Fatigue I or II loading combinations as applicable. In ac a factor of 0.65 (LRFD 3.4.5).

Table 5.13.4-3 Cross-Frame Component Forces

Load Case	Top Chord	Diagonals	Bottom Chord
Intermediate Cross-Frames Simple Spans			
Strength I[1] (kips)	40	70	1.3*RDDP + 85*(S/Dw) – 115 ≥ 100
Fatigue Range[2] (kips)	10	12	0.10*RDDP + 3*(S/Dw) + 5 ≥ 15
Constructibility[3] (kips)	15	10	20
Intermediate Cross-Frames Continuous Spans			
Strength I[1] (kips)	50	0.35*RDDP + 20*(S/Dw) +20 ≥ 70	1.3*RDDP + 85*(S/Dw) – 90 ≥ 100
Fatigue Range[2] (kips)	10	0.05*RDDP +11	0.20*RDDP + 8*(S/Dw) - 2 ≥ 15
Constructibility[3] (kips)	40	20	40
End Cross-Frames			
Strength I[1,4] (kips)	100	100	75
Fatigue Range[2] (kips)	10	15	15
Constructibility[3,4] (kips)	10	10	10

1. The forces shown for Strength I are already factored.
2. Apply load factors for Fatigue I or II loading combinations as applicable. In addition, apply a factor of 0.65 (LRFD 3.4.5).
3. The values are for the unfactored force induced into the cross-frames due to the weight of the concrete deck and SIP forms. Apply other forces effects as applicable.
4. For end cross-frames directly supporting a free edge of the concrete deck, these forces do not include effects of the concrete edge beam weight and a local wheel load.

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AASHTO 10th Edition- CrossFrames

Revisions to improve the prediction of fatigue force ranges in cross-frame members

1. The application of a **factor of 0.65 to the Fatigue I and II** load factors when evaluating cross-frame details for load-induced fatigue. (Article 3.4.5)

2. Specific fatigue truck loading requirements for refined analysis (Article 6.6.1.2.2). The fatigue truck shall be ***confined to one critical transverse position per each longitudinal position*** throughout the length of the bridge in the analysis.

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

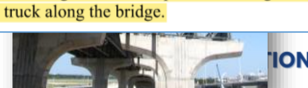
The Fatigue I and II live load factors, γ_{LL}, shall be multiplied by an additional factor of 0.65 when evaluating load-induced fatigue in cross-frames and diaphragms.

6.6.1.2.2—Design Criteria

For load-induced fatigue considerations, each detail shall satisfy:

$$\gamma(\Delta f) \leq (\Delta F)_n \quad (6.6.1.2.2\text{-}1)$$

investigated for load-induced fatigue. To determine the range of force in the cross-frame or diaphragm member under consideration, the single fatigue truck shall be positioned as specified in Article 3.6.1.4.3a, but with the truck confined to a single transverse position during each passage of the truck along the bridge.



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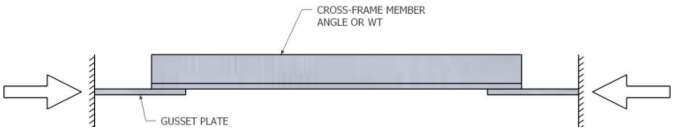
AASHTO 10th Edition- CrossFrames

Modifications to AASHTO Cross-Frame Analysis and Design

3. Revisions to account for cross-frame member **end eccentricity** and for reduced stiffness at the member end connections. In lieu of a more refined analysis, the **equivalent axial stiffness** of single angles and tee-section members should be taken as (Article 4.6.3.3.4) :
   - ***0.65AE*** for non composite (Moved from Commentary to Provisions)
   - ***0.75AE*** for composite (New)

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 $0.65AE$ 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 $0.75AE$ in the analysis model for the composite condition.



Source: https://www.aisc.org/globalassets/nsba/conference-proceedings/2016/horton---2016-wsbs-final.pdf

4. 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 and Article 4.6.3.3.2)

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Bolts, Holes and Connections

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SDG Changes and Updates

5.1.8 Tension Flanges with Holes (LRFD 6.10.1.8)

Add the following to ***LRFD*** 6.10.1.8:

For tension flanges that have cross-frame tab plates with bolt holes that do not remove more than 15% of the gross area of the flange, the holes can be ignored.

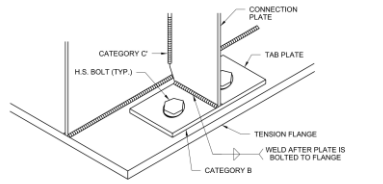
Added a section addressing Tension Flanges with bolt holes for tab plates.

- When a girder meets this criterion, it is then not subject to the stringent criteria that disallow plastic design.

6.10.1.8—Tension Flanges with Holes

When checking flexural members at the strength limit state or for constructibility, the following additional requirement shall be satisfied at all cross-sections containing holes in the tension flange:

$$f_t \le 0.84\left(\frac{A_n}{A_g}\right)F_u \le F_{yt} \qquad (6.10.1.8\text{-}1)$$



Source: NSBA G12.1 Guidelines to Design for Constructability and Fabrication

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SDG Changes and Updates

5.4 BOLTS (LRFD 6.4.3.1 and 6.13.2)

A. Design structural bolted connections as either bearing or slip-critical connections in accordance with ***LRFD*** 6.13.2 and as modified by the following:

1. ***SDG*** 5.13.1 and ***SDG*** 5.14.
2. For composite box girder bridges, design all connections of primary members with slip-critical connections. Secondary members may be designed using bearing connections.
3. Use slip-critical connections for steel integral and/or straddle caps (regardless of their classification of superstructure or substructure).

B. Use standard size holes for all bearing-type connections.

C. Use ASTM F3125 Grade A325 high-strength bolts. Use Type 1 bolts for painted non-weathering steel connections. Use Type 3 bolts for weathering steel connections (uncoated or painted).

D. ~~Do not use~~ ASTM F3125 Grade A490 bolts require approval ~~unless approved~~ by the SSDE and a Technical Special Provision (TSP). The items to be addressed in the TSP include, but are not limited to, the following: handling, lubrication, installation, tightening, testing, and corrosion protection.

Clarified the SSDE approval requirements for A490 bolts, specifying that a TSP is needed and outlining the minimum TSP requirements.

TRANSPORTATION SYMPOSIUM

SDG Changes and Updates

5.11 CONNECTIONS AND SPLICES (LRFD 6.13)

A. Specify and detail bolted (not welded) field connections. ~~A flowchart for the design of bolted field splices is provided in ***Appendix*** 5A.~~ Field welding of sole plates (without sliding surfaces) to the bottom flange of steel I-girders is permissible. Details shall be included in the plans in accordance with ***SDM*** 16.11. Other field welding is allowed only by prior written approval by the SSDE.

C. ~~When the thickness of the plate adjacent to the nut is greater than or equal to 3/4-inch, base the strength of the connection on the bolt shear strength with threads excluded from the shear plane.~~ For cross-frames, the strength of the bolted connections can be based on the bolt shear strength with threads excluded from the shear plane when a Plan Note is included requiring that the washer and direct-tension-indicator (DTI), if used, be placed under the nut and the thickness of the connecting parts (e.g., connection plate, gusset plate, and chord member as applicable), meet the following:

*Commentary: The requirements for excluding threads ~~for cross-frame connections~~ are ~~based on criteria~~ contained in **LRFD** C6.13.2.7. ~~the upcoming 10th Edition of the **AASHTO LRFD Bridge Design Specifications** which provides for a significant reduction in the number of bolts when threads can be excluded. The 3/4-inch plate thickness criteria will be retained until the Department adopts the 10th edition.~~*

Remove the reference to Appendix 5A. Revised and removed language is now available in the LRFD 10th edition. Reference LRFD article 6.13.2.7 about shear planes with the threads included/excluded

AASHTO 10th Edition-High-Strength Bolts

Shear Resistance of High-Strength Bolts – Threads Included or Excluded

- Shear planes located in the ***transition length*** of high-strength bolts should be considered with the threads included (Article 6.13.2.7).
- Such shear planes have traditionally been treated as if the threads were excluded from the shear plane
- 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).

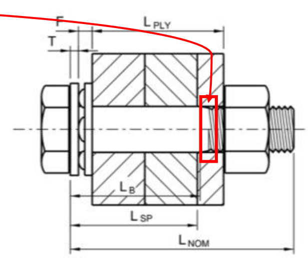


Figure C6.13.2.7-1—Dimensions for Determination of Threads Excluded from or Included in the Shear Plane

TRANSPORTATION SYMPOSIUM

AASHTO 10th Edition-Slip-Critical vs. Bearing

Slip-Critical vs. Bearing-Type Connections for Bracing Members

- Joints of cross-frame or diaphragm members 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 if slip occurs, it is not anticipated to be detrimental to the geometry or serviceability of the structure.
- Faying surfaces of joints that are designed as bearing-type connections **need not satisfy** the surface condition preparation for slip-critical connections (specified in Article 6.13.2.8); coatings are permitted in bearing-type connections.

TRANSPORTATION SYMPOSIUM

SDG and SDM 2025

Connections for Bracing Members

5.4 BOLTS (LRFD 6.4.3.1 and 6.13.2)

A. Design structural bolted connections as either bearing or slip-critical connections in accordance with ***LRFD*** 6.13.2 and as modified by the following:

1. ***SDG*** 5.13.1 and ***SDG*** 5.14.

Table 5.13.1-1 Design Criteria for Straight Steel I-Girder Units

Case	SDG Section	Skew Angle (θ)	Skew Index (I_s)	Cross-Frame Configuration	Required Method(s) of Analysis[3]	Cross-Frame Connection Type	Calculate Cross-Frame Rating Factor
1[1]	5.13.2 & 5.13.3	$\theta \le 20°$	$I_s \le 0.45$	contiguous[2] and parallel to skew angle	LGA [*LRFD* 4.6.2.2]	Bearing	No
2[1]	5.13.2 & 5.13.4	$20° < \theta \le 50°$	$I_s \le 0.3$	contiguous[2] and normal to girders	LGA [*LRFD* 4.6.2.2]	Bearing	No
3	5.13.5	$\theta \le 50°$	$I_s \le 0.45$	Any[5]	LGA and RMA [*LRFD* 4.6.2.2 & 4.6.3]	Bearing	No
4	5.13.6	$\theta \le 50°$	$I_s \le 0.6$	Any[5]	RMA [*LRFD* 4.6.3]	Slip-critical	No[6]
5	5.13.7	$\theta \le 60°$	Any	Any[5]	RMA 3D-FEA [*LRFD* 4.6.3]	Slip-critical	Yes

5.14 HORIZONTALLY CURVED STEEL I-GIRDER UNITS

Design horizontally curved steel I-girder units in accordance with ***LRFD*** and the following requirements:

A. For horizontally-curved I-girder units satisfying all the conditions specified in ***LRFD*** 4.6.1.2.4b, design the cross-frame connections as bearing connections. Do not calculate a load rating factor for the intermediate or end cross-frames.

B. For horizontally-curved I-girder units not satisfying all the conditions specified in ***LRFD*** 4.6.1.2.4b, design the cross-frame connections as slip-critical connections. Do not calculate a load rating factor for the intermediate or end cross-frames except as noted in Number C.1 below.

5.3 TYPICAL STEEL GENERAL NOTES (SDM)

E. Bolted Connections

3. For cross-frame connections identified as bearing, install high-strength bolts in accordance with Section 460-5 of the Specifications. Prepare faying surfaces to SSPC SP-10. Threads are excluded from the shear plane. The washer and direct-tension-indicator (DTI), if used, must be placed under the nut. Do not place a washer or DTI under the bolt head. [Use as applicable. See ***SDG*** 5.4, ***SDG*** 5.11.1, ***SDG*** 5.13, ***SDM*** 16.7, and ***SDM*** 16.8.]

Stability Requirement

TRANSPORTATION SYMPOSIUM

SDG Changes and Updates

5.13.2 Design Criteria for Case 1 and 2

This section covers the criteria for both Case 1 and 2. For additional requirements for Case 1 or Case 2, see ***SDG*** 5.13.3 and ***SDG*** 5.13.4, respectively.

A. Meet the supplemental conditions to ***LRFD*** 4.6.2.2 as shown in Table 5.13.2-1. When the number of girders equals three, the bridge must meet the same range of applicability for S, t_s, and L as stated for four or more girders.

Table 5.13.2-1 Supplemental Conditions to LRFD 4.6.2.2

Supplemental Conditions[1]	LRFD 4.6.2.2 Conditions
1. Width of deck can vary up to 5 degrees[2]	Width of deck is constant
2. Girder spacing can be non-parallel up to 5 degrees[2,3]	Girders are parallel
3. The beam spacing must meet the range of applicability	For beam spacing exceeding the range of applicability as specified in tables in Articles 4.6.2.2.2 and 4.6.2.2.3, the live load on each beam is based on the lever rule.
4. $10{,}000 \le K_g \le 10{,}000{,}000$	$10{,}000 \le K_g \le 7{,}000{,}000$
5. Difference between skew angles of two adjacent supports does not exceed 10 degrees	Not explicitly addressed
6. $d_e / S \le 0.35$	Not explicitly addressed
7. $0.95 \le S/D_W \le 2.00$	Not addressed
8. RDDP < 175 (see ***SDG*** 5.13.2.K)	Not addressed

M. For intermediate cross-frames, use only load combinations for Strength I, Fatigue I or II, and Constructability. Also meet the stability bracing requirements of ***LRFD*** 6.7.4.2.2. Use "K" frames for intermediate cross-frames as shown in ***SDM*** Figure 16.7-1. Evenly space cross-frames, with a spacing not to exceed 30-feet.

Added language to meet the stability bracing requirements of LRFD

Added language to indicate that 3 girders must meet the same range of applicability for S, L, and t_s as stated for 4 girders.

TRANSPORTATION SYMPOSIUM

SDG Changes and Updates

5.13.5 Design Criteria for Case 3

A. Design the girders using a LGA with the LLDF per ***LRFD*** 4.6.2.2 using the criteria in ***SDG*** 5.13.2 A through J. The supplemental conditions 5, 6, 7, and 8 of Table 5.13.2-1 do not apply for Case 3 bridges. Use the use the multiplicative factors in ***SDG*** 5.13.4.A, except for the bearing reactions.

B. Use an RMA to determine fatigue stress ranges, bearing reactions, flange lateral bending stresses, girder cambers, and cross-frame forces.

C. For design of the intermediate cross-frames, use only Load Combinations for Strength I, Fatigue I or II, and Constructability. Also meet the stability bracing requirements of ***LRFD*** 6.7.4.2.2.

D. For end cross-frames located at the end of a unit below a free edge of the concrete deck and at interior supports located below a continuous concrete deck, use only load combinations for Strength I, Strength III, Fatigue I or II, and Constructability.

E. If the requirements of this Section cannot be met, use Case 4 criteria.

Added language to meet the stability bracing requirements of LRFD

TRANSPORTATION SYMPOSIUM

AASHTO 10th Edition-Minimum Stability Bracing

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

$$(\beta_T)_{act} \geq (\beta_T)_{req} \quad (6.7.4.2.2\text{-}1)$$

In lieu of a more refined analysis, the actual overall stiffness of the torsional bracing system, $(\beta_T)_{act}$, shall be calculated as follows:

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

$(\beta_T)_{req}$ = 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:

$$= \frac{2.4L}{\phi_{sb} n E I_{yeff}} \left(\frac{M_u}{C_b}\right)^2 \quad (6.7.4.2.2\text{-}2)$$

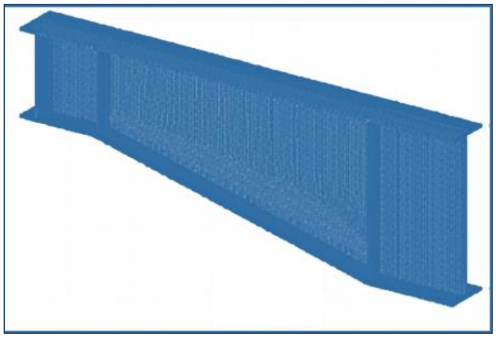
- Otherwise:

$$= \frac{3.6L}{\phi_{sb} n E I_{yeff}} \left(\frac{M_u}{C_b}\right)^2 \quad (6.7.4.2.2\text{-}3)$$

β_g = effective in-plane girder stiffness for stability bracing (kip-in./rad.)
β_{sec} = cross-sectional (girder web) distortion stiffness for stability bracing (kip-in./rad.)
β_{br} = brace stiffness of the diaphragm or cross-frame that restrains twist of a beam or girder (kip-in./rad.)

TRANSPORTATION SYMPOSIUM

Tapered Girder



TRANSPORTATION SYMPOSIUM

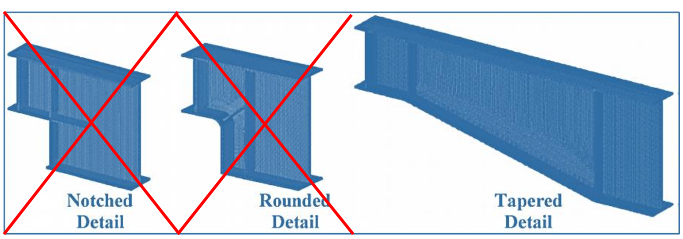
SDG Changes and Updates

5.1.3 Dapped or Tapered Girder Ends at Simple Supports

Dapped steel box girders or dapped steel plate girders are not permitted. Girder end tapers with a maximum slope of 1:12 are permitted with SSDE approval.

Commentary: Dapped beam details are inherently prone to cracking due to the abrupt change in cross-section and the resulting complex flow of internal stresses.

We now allow tapered girder ends with a slope of 1:12 or less with SSDE approval.
While dapped ends are prohibited due to poor fatigue performance, tapered ends at a low slope do not share this issue and are not considered risky.



Resource: FHWA/TXDOT-Report 0-2102-1-Project 0-2102

This is for simple support, different than the haunch at pier/negative moment

TRANSPORTATION SYMPOSIUM

Pedestrian Bridges

We have a comprehensive presentation on this topic. Make sure to attend Dennis's presentation this afternoon (2:05 PM - 3:00 PM)

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SDG Changes and Updates

2.10 REDUNDANCY AND OPERATIONAL IMPORTANCE (LRFD 1.3.4 AND 1.3.5)

Use redundant (multiple-load-path) superstructure systems unless otherwise permitted by ***SDG*** 4.1.A or ***SDG*** 5.1.D.

A. Redundancy (LRFD 1.3.4)

Delete the Redundancy Factors for the strength limit state, η_R, in ***LRFD*** 1.3.4 and use η_R = **1.0** except as defined below:

Table 2.10-1 Redundancy Factors

Component	η_R Factor
Steel I-Girders in Two Girder Cross Sections[1]	1.20
Concrete I-Beams in Two Beam Cross Sections[2]	1.10
Truss/Arch Bridges (excluding steel pedestrian trusses meeting the Category 1 conditions of FDM 266.4)	1.20
Steel Floor beams with Spacing > 12-feet and Non-Continuous Deck[3]	1.20

Removed the redundancy factor requirement for prefabricated steel truss pedestrian bridges classified as Category 1.

We are easing the requirements for fabricators of prefabricated steel two-truss systems (using steel hollow structural sections) that comply with FDM 266.4. These systems are classified as **Category 1 and Redundant**.

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Fatigue and Fracture

SDG Changes and Updates

5.3.2 Fracture (LRFD 6.6.2)

C. Nonredundant Steel Tension Members~~Fracture critical members~~ are defined as tension members or tension components of nonredundant members whose failure would result in the collapse of the structure. This includes but is not limited to the following:

1. All tension components of two I-girder superstructures.
2. All tension components in the positive moment region of two box superstructures. Negative moment regions over the piers have four top flanges and are therefore considered redundant.
3. All tension components of straddle and integral piers.
4. All tension components of a two-truss superstructure, expect those classified as Category 1 (refer to *FDM* 266.4).

Added language to classify all tension components of a two-truss superstructure as NSTM, except for Category 1 pedestrian bridges

SDG Changes and Updates

5.3.3 Fatigue (LRFD 6.6.1.2)

Use ***LRFD*** Table 6.6.1.2.3-1 to reference Detail Categories referred to below.

A. In addition to ***LRFD*** 6.6.1.2.3, components and details on longitudinal primary members having Detail Categories A, B, B', C and C' must meet the Fatigue I limit state.

B. Do not use Detail Category E or E' ~~as defined in ***LRFD*** Table 6.6.1.2.3-1~~. Category E' welds are allowed for use in cross-frame connections.

C. For Nonredundant Steel Tension Member (NSTM)~~Fracture Critical Members (FCM)~~, use fatigue details classified as Detail Category C or better (except for Note D below) ~~as defined in ***LRFD*** Table 6.6.1.2.3-1.~~ For steel truss bridges, submit details for SSDE approval.

D. Use Detail Category D for drain and ventilation (vent) holes required by ***SDG*** 5.6.2.

For NSTM, use Fatigue Detail Category C or better (except Category D for drain and vent holes of Box Sections)

For NSTM steel truss bridges (Vehicular and Pedestrian), except for Category 1 Pedestrian bridges, submit the fatigue details for SSDE approval.

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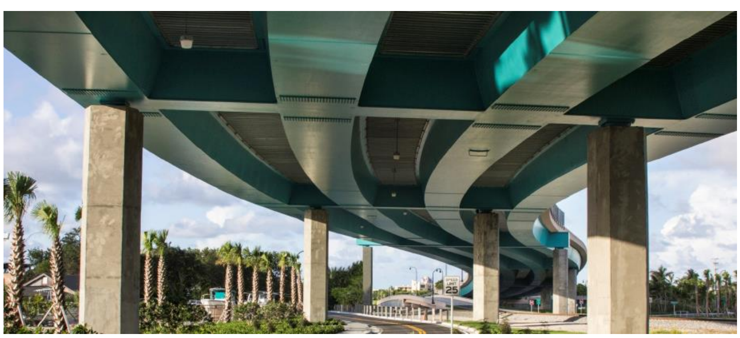
Box Section



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SDG Changes and Updates

5.6 BOX SECTIONS

5.6.2 Access and Maintenance

D. Other Exterior Openings:

1. Design each box girder with minimum 2-inch diameter ventilation (vent) and 2-inch diameter drain holes ~~or drain holes located in the bottom flange on both sides of the box spaced at approximately 50- feet or as needed to provide proper drainage. Place drains at all low points against internal barriers.~~
   a. Place drain holes at the low side of the bottom flange located at each end bent, pier, and the low point of the unit, as applicable. L~~At a minimum, l~~ocate drain holes approximately 3'-0" from bearing stiffeners. Place drain holes at all low points against internal barriers.
   b. Place vent holes in both webs of each box spaced at approximately 50'-0" along the entire girder length. Locate vent holes ¼ to ½ of the web height from the top flanges.

Source: NSBA G12.1 Guidelines to Design for Constructability and Fabrication

Updated drain and vent holes requirements for steel box girders. Limit the # of drains (Category D details) in the bottom flange (high stress range) to bent, pier, and low points, and add vent holes (Category D details) at the web (low stress range)

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SDM Changes and Updates

16.11 MISCELLANEOUS DETAILS

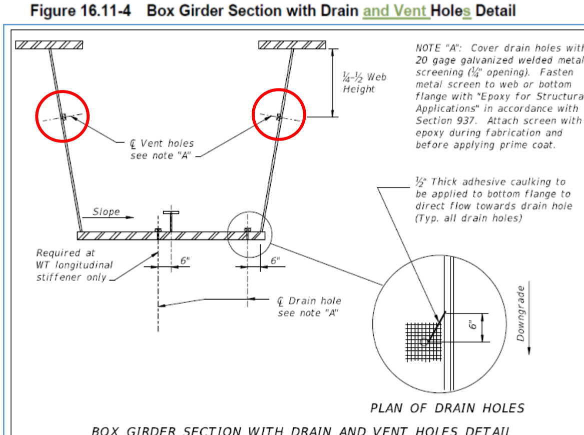
Steel girder details shall provide sufficient information for fabrication and erection of the girders. These miscellaneous details can be shown on a separate sheet or incorporated in any of the previous sheets, as space permits. The following details are typical for most steel girder applications:

A. Transverse shear connector spacing detail (weld symbol not required in this detail as it is covered by ***Specifications*** Section 502). See Figure 16.11-3.

E. Box girder details including the following:

1. Access opening details.
2. Vermin guards.
3. Diaphragm access opening details.
4. Drain holes and vent holes with screen covers~~. Drain holes should be a minimum of 5 feet from centerline pier or front face of backwall~~. See Figure 16.11-4.
5. Electrical access holes.
6. Longitudinal stiffener details. Detail longitudinal stiffener termination as shown. See Figure 16.11-1.

Figure 16.11-4 Box Girder Section with Drain and Vent Holes Detail



Updated Figure 16.11-4 for drain hole and vent hole. Place vent holes in both webs of each box spaced at 50' along the entire girder length. Locate vent holes ¼ to ½ of the web height from the top flanges

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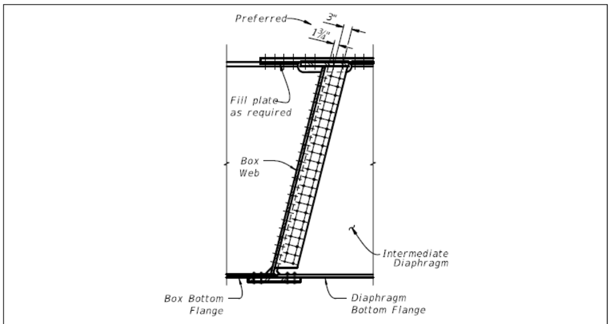
SDM Changes and Updates

Figure 16.8-5 Steel Box Girder Intermediate Diaphragm Connection



Commentary: The bottom flanges of permanent external diaphragms must be connected to the box girder ~~bottom flanges as shown in Figure 16.8-5~~. Otherwise, the forces from the external diaphragm will induce out-of-plane distortions which may cause a fatigue crack.

Removed commentary language that limited external diaphragm connection options. Give more flexibility and alleviate issues with installation and conflicts with the bearings

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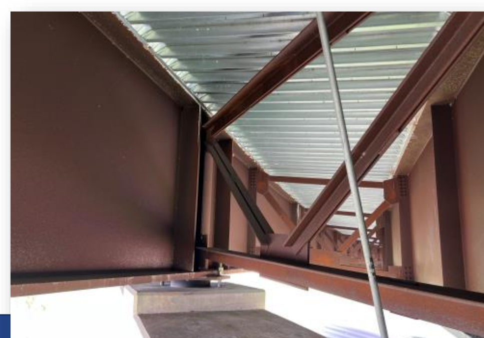
Half Round Bearing Stiffener (HRBS)



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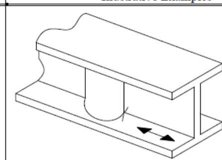
AASHTO 10th Edition

Half-Round Bearing Stiffeners (HRBS)

- HRBS are to be **fillet welded to both flanges** of the cross section
- All welds should be continuous **to seal the interior** of the half-round.
- The **connection plates** are to be fillet welded to the half-rounds but **should not be attached to the flanges** of the cross-section.

AASHTO Table 6.6.1.2.3-1
New Condition 4.2: Category C'
Fatigue characterization of half-round bearing stiffeners

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	

SDG Changes and Updates

5.9 BEARING STIFFENERS (LRFD 6.10.11.2)

A. For plate girder bridges with grades greater than 4%, require bearing stiffeners to be vertical under full dead load. For grades less than or equal to 4% bearing stiffeners may be placed normal to the bottom flange. In this case, the effect of the grade must be considered in design of the stiffener.

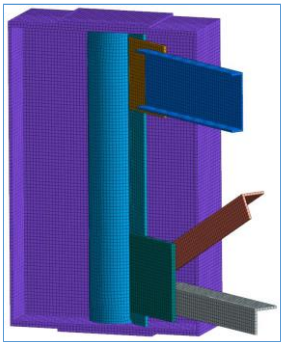
F. Half-round bearing stiffeners require prior approval of the SSDE. Contact the SSDE for additional design guidance.

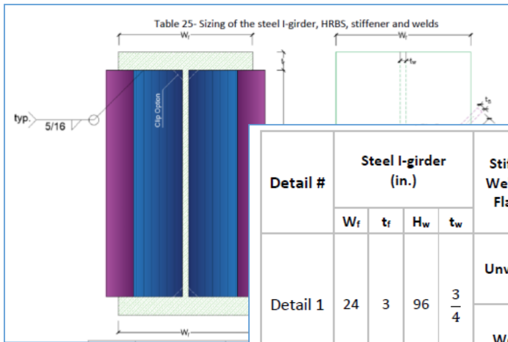
Modification for Non-Conventional Projects:
Delete *SDG* 5.9.F and replace with the following:
F. Half-round bearing stiffeners are not allowed.

Added SSDE approval requirement for HRBS. SDO is currently conducting research and has not established final conclusions on the adoption

- Fabricated by pipe splitting or plate rolling.
- Allow for a perpendicular connection between the stiffener and a cross-frame for any skew angle at a support
- Offers simpler crossframe fabrication, installation, and fit-up due to the elimination of bent plates
- Provide increased warping torsional restraint for LTB at the ends of unbraced lengths

SDG 2027- FDOT HRBS Updates





FDOT –FIU Research Recommendations regarding dimensions, thicknesses, and welding details

Detail #	Steel I-girder (in.)				Stiffener Welded to Flanges	Clipping Type	PL-A (HRBS) (in.)			PL-B (in.)			
	W_f	t_f	H_w	t_w			D	R	t_A	L	B	t_B	α
Detail 1	24	3	96	$\frac{3}{4}$	Unwelded	Non-Clipped	16	8	3/4	94	8	1	57°
						Clipped							
					Welded	Non-Clipped			1/2	96	8	1	57°
						Clipped							
Detail 2	22	$2\frac{5}{8}$	90	$\frac{5}{8}$	Unwelded	Clipped	16	8	5/8	87	8	1	54°
					Welded	Clipped			3/8	90			
Detail 3	16	2	66	$\frac{9}{16}$	Unwelded	Clipped	11	5.5	5/8	63	8	1	23°

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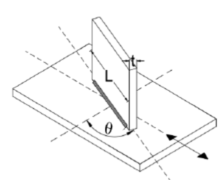
Fatigue characterization of Obliquely Oriented Welded Attachments

AASHTO Table 6.6.1.2.3-1
New Condition 7.3: fatigue categories transitioning between C' and E as a function of the skew angle, θ

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
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	
$\theta \le 20°$	C'	44×10^8	3	12	975		
$20° < \theta \le 30°$	C	44×10^8	3	10	1680		
$30° < \theta \le 45°$	D	22×10^8	3	7	2450		
$45° < \theta < 90°$	E	11×10^8	3	4.5	4615		

(for attachments longer than 4 inches and less than 1-inch thick attached by groove or fillet welds)

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AASHTO 10th Edition- Fatigue Detail Table

Revised the fatigue detail categories Table 6.6.1.2.3-1:

- Changed the exponent for the finite-life fatigue resistance equation 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 75-year $(ADTT)_{SL}$ equivalent to infinite life to Table 6.6.1.2.3-1
- Eliminated Tables 6.6.1.2.3-2, 6.6.1.2.5-1, and 6.6.1.2.5-3

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{m}} \qquad (6.6.1.2.5\text{-}2)$$

in which:

$$N = (365)(75)n(ADTT)_{SL} \qquad (6.6.1.2.5\text{-}3)$$

Table 6.6.1.2.3-2—75-year $(ADTT)_{SL}$ Equivalent to Infinite Life

Detail Category	75-year $(ADTT)_{SL}$ Equivalent to Infinite Life (trucks per day)
A	690
B	1120
B′	1350
C	1680
C′	975
D	2450
E	4615
E′	8485

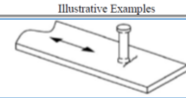
Table 6.6.1.2.5-3—Constant-Amplitude Fatigue Thresholds

Detail Category	Threshold (ksi)
A	24.0
B	16.0
B′	12.0
C	10.0
C′	12.0
D	7.0
E	4.5
E′	2.6
ASTM F3125/F3125M, Grades A325 and F1852 Bolts in Axial Tension	31.0
ASTM F3125/F3125M, Grades A490 and F2280 Bolts in Axial Tension	38.0

Table 6.6.1.2.5-1—Detail Category Constant, A

Detail Category	Constant, A (ksi^3)
A	250.0×10^8
B	120.0×10^8
B′	61.0×10^8
C	44.0×10^8
C′	44.0×10^8
D	22.0×10^8
E	11.0×10^8
E′	3.9×10^8
ASTM F3125/F3125M, Grades A325 and F1852 Bolts in Axial Tension	17.1×10^8
ASTM F3125/F3125M, Grades A490 and F2280 Bolts in Axial Tension	31.5×10^8

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
9.1 Base metal at shear connectors attached by fillet or automatic stud welding. Use the fatigue live load stress range, Δf, and Eq. 6.6.1.2.2-1.	C	44×10^8	3	10	1680	At the toe of the weld in the base metal	

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AASHTO 10th Edition- Shear Stud Design

- Revised the equation for the nominal shear resistance, Q_n, of a stud shear connector at the strength limit state (somewhat more conservative).

6.10.10.4.3—Nominal Shear Resistance

The nominal shear resistance of one stud shear connector embedded in a concrete deck shall be taken as:

$$Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} \leq A_{sc} F_u \quad (6.10.10.4.3\text{-}1)$$

→

6.10.10.4.3—Nominal Shear Resistance

The nominal shear resistance of one stud shear connector embedded in a concrete deck shall be taken as:

$$Q_n = 0.70 A_{sc} F_u \quad (6.10.10.4.3\text{-}1)$$

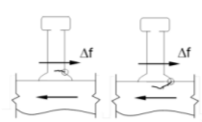
- Deleted all references to channel shear connectors.

AASHTO 10th Edition- Shear Stud Design

Revised the fatigue detail Table 6.6.1.2.3-1 as follows:

- Added the fatigue resistance data for studs and high-strength bolts to Table 6.6.1.2.3-1, with new condition 9.2
- Changed the slope of the fatigue resistance curve for studs in the finite-life region from -3.00 to a flatter slope of -5.00.

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

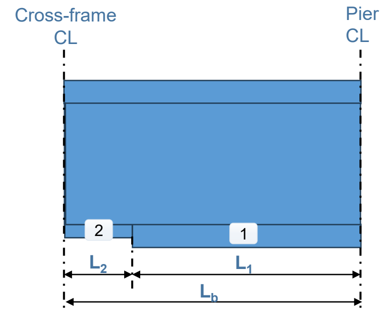
Description	Category	Constant A (ksi^m)	Fatigue Growth Constant, m	Threshold $(\Delta F)_{TH}$ ksi	75-year $(ADTT)_{SL}$ Equivalent to Infinite Life (trucks per day)	Potential Crack Initiation Point	Illustrative Examples
9.2 Shear connectors or base metal at shear connectors attached by fillet or automatic 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	

TRANSPORTATION SYMPOSIUM

AASHTO 10th Edition-LTB

Lateral Torsional Buckling of Nonprismatic Unbraced Lengths

- Replace the approximate approach with more accurate 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 LTB resistance of **longer nonprismatic** unbraced lengths of **noncomposite I-section** members during **temporary construction conditions**.



Girder Elevation

AASHTO 10th Edition-LTB

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.

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.

- The methods do not supersede each other.
- There is no favor given to any of the alternative methods to calculate γ_e

AASHTO 10th Edition-LTB

- Revision to LRFD Articles 6.10.8.2.3b, replacement of the 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.

- Revision to LRFD Articles 6.10.8.2.3a, replacement of the equation for the compact unbraced length limit, Lp, with the equation given in the AISC Specification for general I-section members:

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

9th edition

$$L_p = 1.0\,r_t\sqrt{\frac{E}{F_{yc}}}$$

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Specs 460 and 962 Update

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Specs 460 Updates

Specs 460: Structural Steel and Miscellaneous Metals

Summary of Key Changes:

- Terminology updates (primary members and NSTM) are necessary to align with AASHTO and SDG
- Section 460-1.2 is removed due to fabricator qualification requirements explained in Materials Manual 11.1 Volume II.
- Reference to A490 bolts is being removed from the Specifications.
  - The bolting requirements in Section 460 are explicitly for A325 bolts.
  - A TSP is required for A490 bolts.
- Updating bolt hole geometry to align with AASHTO, namely 1” and 1 1/8”

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Specs 962 Updates

Specs 962: Structural Steel and Miscellaneous Metal Items (Other Than Aluminum)

Summary of Key Changes:

- Terminology updates (primary member and NSTM) are necessary to align with AASHTO and SDG.
- Reference to A490 bolts is being removed from the Specifications.
- Updated the tables to reflect the corrected ASTM standards for hardware (bolts, nuts and washers) and removed inconsistencies.
- Updated the galvanizing table for hardware.

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Safety Message

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