Updates to the AASHTO Design Specification

LRFD BDS Section 6
8th Ed. (2017) & 9th Ed. (2020)

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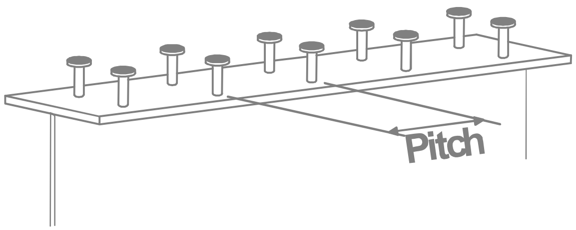
Significant Updates in the 8th Edition LRFD BDS (Section 6)

- Increase in Maximum Shear Connector Spacing
- Introduction of the Unified Effective Width Approach
- Recommended Details to Avoid Conditions Susceptible to Constraint-Induced Fracture
- Increase in the Fatigue Load Factors
- Skewed and Curved I-Girder Bridge Fit & Framing Arrangements
- Primary vs. Secondary Members, Charpy Requirements, FCMs & SRMs

Increase in Maximum Shear Connector Spacing



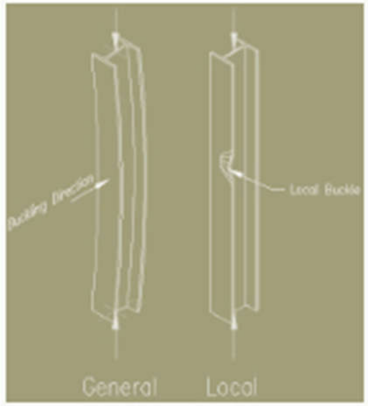
Description of Specification Revisions:

- Increased maximum shear connector spacing (pitch) from 24.0 inches to 48.0 inches. However, only for web depths > 24.0 inches.

Introduction of the Unified Effective Width Approach

Description of Specification Revisions:

- Introduced the unified effective width approach for the calculation of the nominal compressive resistance of members with slender element cross-sections (Articles 6.9.4.1 & 6.9.4.2).
  - Adopted in the 2016 AISC Specification and the 2016 AISI *North American Specification for the Design of Cold-Formed Steel Structural Members.*
  - Accounts for the effect of potential local buckling of slender elements, supported along one *or* two longitudinal edges, on the overall column-buckling resistance of the member.



Introduction of the Unified Effective Width Approach

- Replaced the previous Q-factor approach to handle compression members with slender elements – originally adopted in the 1969 AISC and AISI Specifications.
- Table 6.9.4.2.1-1 was revised to replace the "plate-buckling coefficients", *k*, with corresponding width-to-thickness ratio limits, λ_r.

6.9.4.2.1-1—~~Plate Buckling Coefficients and Width of Plates for Axial Compression~~ Element Width-to-Thickness Ratio Limits and Element Widths for Axial Compression

Plates Supported along One Longitudinal Edge ~~(Unstiffened Elements)~~	~~k~~ λ_r	b
Flanges of Rolled I-, Tee, and Channel Sections; Plates Projecting from Rolled I-Sections; and Outstanding Legs of Double Angles in Continuous Contact	~~0.56~~ $0.56\sqrt{\frac{E}{F_y}}$	• Half-flange width of rolled I- and tee sections • Full-flange width of channel sections • Distance between free edge and first line of bolts or welds in plates

- Reference to the terms "unstiffened elements" and "stiffened elements" was removed in the specification and commentary.

Introduction of the Unified Effective Width Approach

- The nominal compressive resistance, P_n, is obtained by multiplying F_{cr} based on the gross cross-sectional area by an effective area, A_{eff}.
- A_{eff} is generally computed as the summation of effective areas of the cross-section based on reduced effective widths, b_e, for each slender element in the cross-section (Article 6.9.4.2.2a).

$$b_e = b\left(1 - c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} \qquad (6.9.4.2.2a\text{-}2)$$

in which:

c_1 ≡ effective width imperfection adjustment factor determined from Table 6.9.4.2.2a-1
c_2 ≡ effective width imperfection adjustment factor determined from Table 6.9.4.2.2a-1

$$= \left(1 - \sqrt{1 - 4c_1}\right) / \left(2c_1\right) \qquad (6.9.4.2.2a\text{-}3)$$

F_{el} ≡ elastic local buckling stress (ksi)

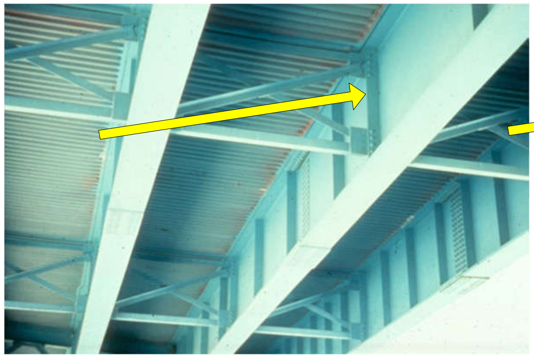
$$\equiv \left(c_2 \frac{\lambda_r}{(b/t)}\right)^2 F_y \qquad (6.9.4.2.2a\text{-}4)$$

- For circular tubes and round HSS, A_{eff} is computed directly from equations (Article 6.9.4.2.2b).

Recommended Details to Avoid Conditions Susceptible to Constraint-Induced Fracture

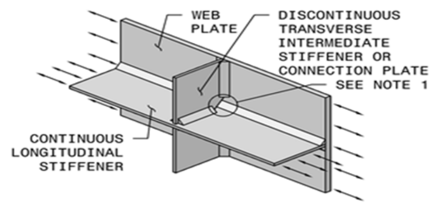
Description of Specification Revisions:

- New definitions are added to Article 6.2 for a:
  - ‘Transverse Connection Plate’
  - ‘Lateral Connection Plate’

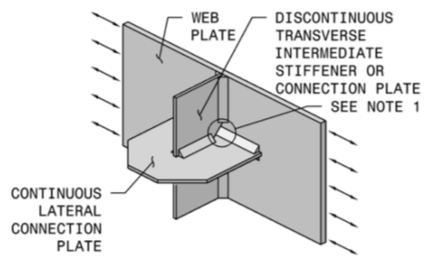


Recommended Details to Avoid Conditions Susceptible to Constraint-Induced Fracture

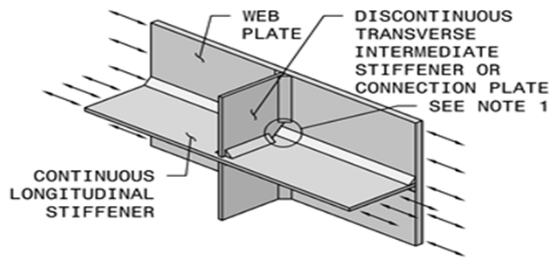
- Two new tables are added to Article 6.6.1.2.4 providing recommended details to avoid conditions susceptible to constraint-induced fracture in regions subject to a net tensile stress under Strength I.
  - Table 6.6.1.2.4-1:



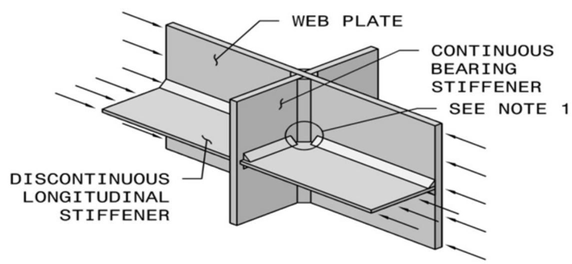
  - Table 6.6.1.2.4-2:



Recommended Details to Avoid Conditions Susceptible to Constraint-Induced Fracture



Note 1: If a gap is specified between the weld toes, the recommended minimum distance between the weld toes is ¾ in., but shall not be less than ½ in. Larger gaps are also acceptable.



Increase in the Fatigue Load Factors

Description of Specification Revisions:

- The Fatigue I load factor was changed from 1.50 to 1.75 and the Fatigue II load factor was changed from 0.75 to 0.80. The commentary was revised accordingly to explain the changes.

Fatigue I— *LL, IM & CE* only	—	~~1.50~~ 1.75	—
Fatigue II— *LL, IM & CE* only	—	~~0.75~~ 0.80	—

Increase in the Fatigue Load Factors

- The values in Table 6.6.1.2.3-2 and Equation C6.6.1.2.3-1 were changed to accommodate the revised load factors.

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

Detail Category	75-yrs $(ADTT)_{SL}$ Equivalent to Infinite Life (trucks per day)
A	~~530~~ 690
B	~~860~~ 1120
B′	~~1035~~ 1350
C	~~1290~~ 1680
C′	~~745~~ 975
D	~~1875~~ 2450
E	~~3530~~ 4615
E′	~~6485~~ 8485

$$\xcancel{75\_Year(ADTT)_{SL} = \frac{A}{\left[\frac{(\Delta F)_{TH}}{2}\right]^3 (365)(75)(n)}}$$

$$75 - Year(ADTT)_{SL} = \frac{A}{\left[\frac{0.80(\Delta F)_{TH}}{1.75}\right]^3 (365)(75)(n)} \qquad \text{(C6.6.1.2.3-1)}$$

Increase in the Fatigue Load Factors

- Table 6.6.1.2.5-2 was revised to remove the increase in number of stress cycles per truck passage for spans ≤ 40 feet.

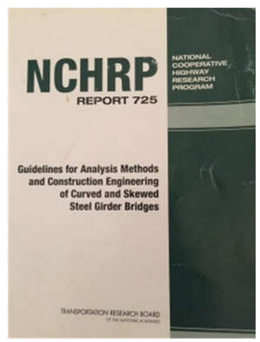
Table 6.6.1.2.5-2—Cycles per Truck Passage, *n*

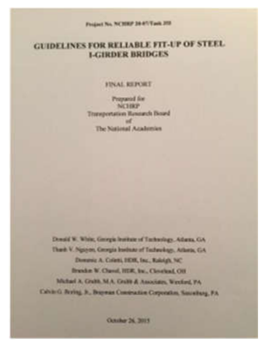
Longitudinal Members	~~Span Length~~	
	~~>40.0 ft~~	~~≤40.0 ft~~
Simple Span Girders	1.0	~~2.0~~
Continuous Girders		
1) near interior support	1.5	~~2.0~~
2) elsewhere	1.0	~~2.0~~
Cantilever Girders	5.0	

- Slight increase in weight of continuous-span girders due to increase in bottom-flange size in positive moment regions.
- Some additional shear connectors will likely be required.

Skewed and Curved I-Girder Bridge Fit & Framing Arrangements

- NCHRP Project 12-79: “Guidelines for Analysis Methods and Construction Engineering of Curved and Skewed Steel Girder Bridges”
- NCHRP Project 20-07, Task 355: “Guidelines for Reliable Fit-Up of Steel I-Girder Bridges”





Skewed and Curved I-Girder Bridge Fit & Framing Arrangements

Description of Specification Revisions:

- The contract documents should state the fit condition for which the cross-frames or diaphragms are to be detailed for the following I-girder bridges (Article 6.7.2):

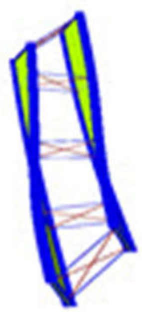
  - Straight bridges where one or more support lines are skewed more than 20 degrees from normal;
  - Horizontally curved bridges where one or more support lines are skewed more than 20 degrees from normal and with an L/R in all spans less than or equal to 0.03; and
  - Horizontally curved bridges with or without skewed supports and with a maximum L/R greater than 0.03.

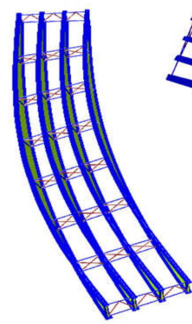
L = span length bearing to bearing along the centerline of the bridge

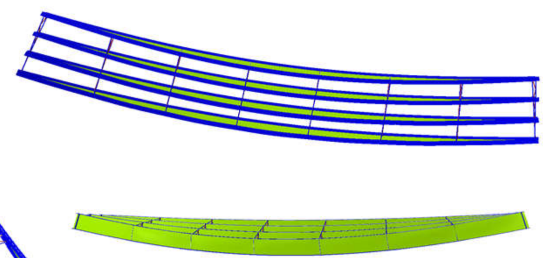
R = radius of the centerline of the bridge cross-section

Skewed and Curved I-Girder Bridge Fit & Framing Arrangements

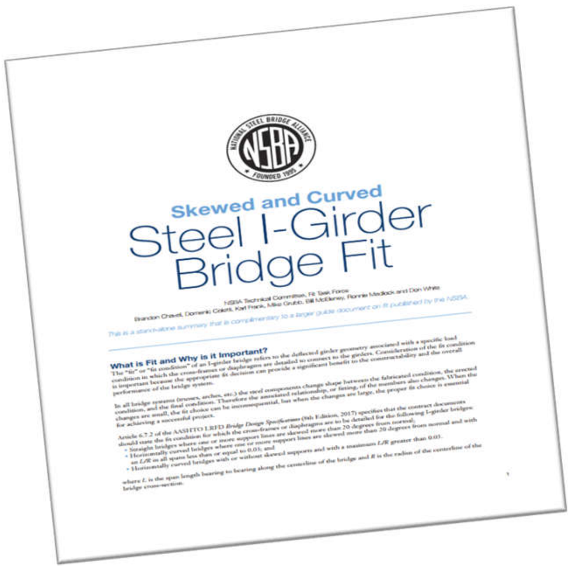
Fit Condition – deflected girder geometry associated with a targeted dead load condition for which the cross-frames are detailed to connect to the girders.







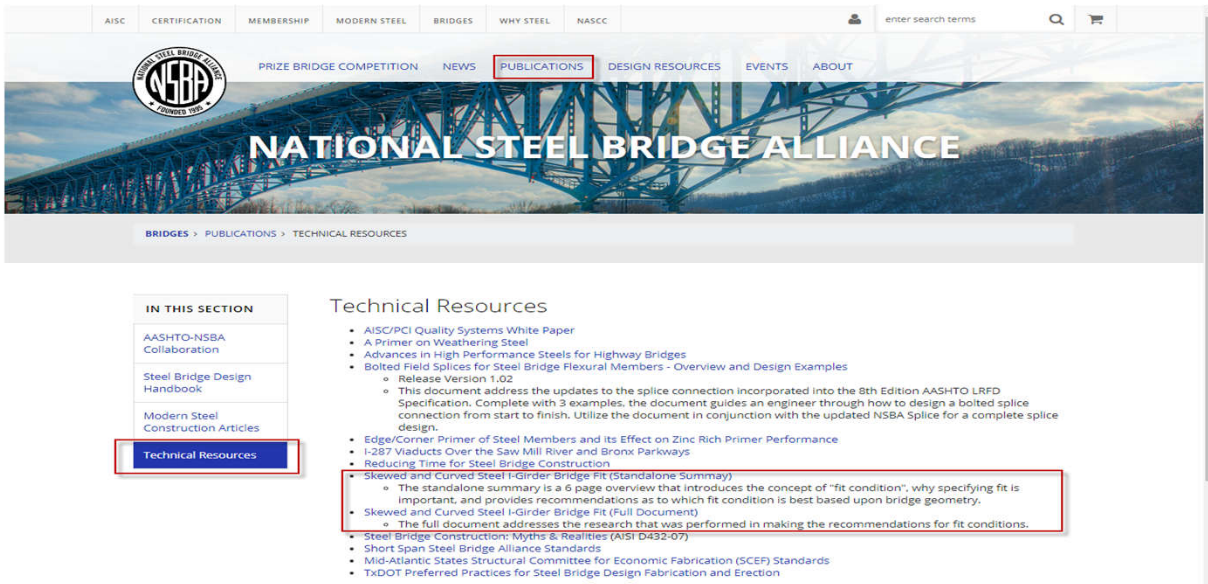
Skewed and Curved I-Girder Bridge Fit & Framing Arrangements



Summary

Full Document

Skewed and Curved I-Girder Bridge Fit & Framing Arrangements



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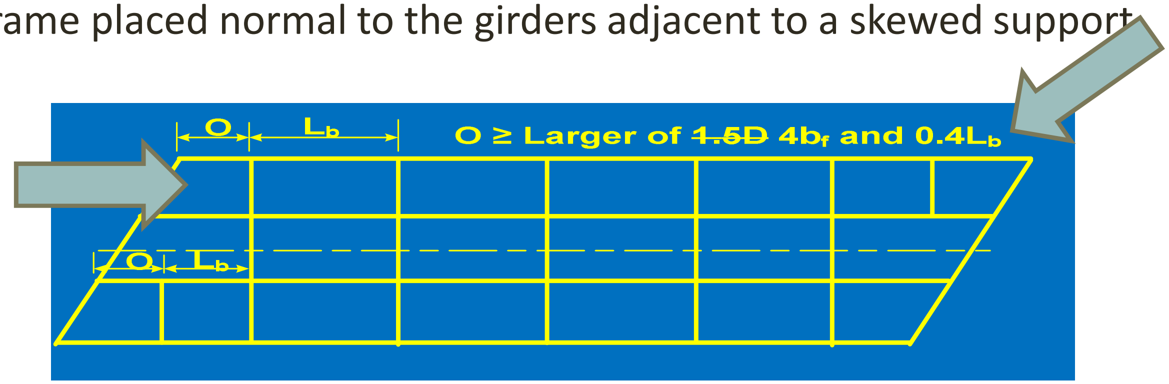
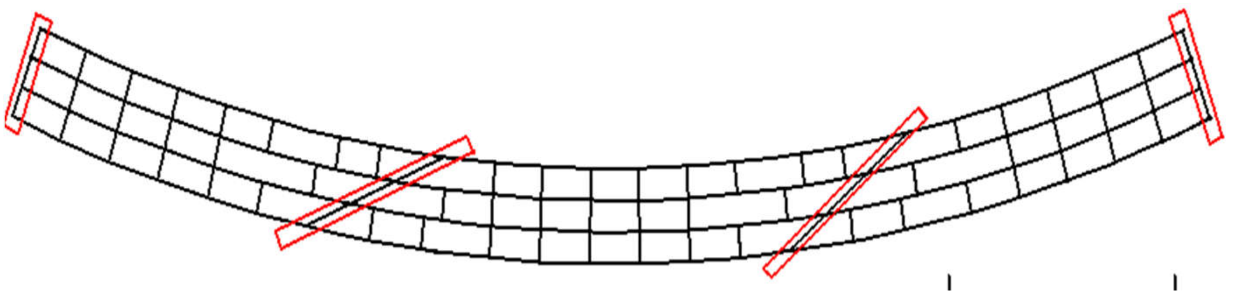
Skewed and Curved I-Girder Bridge Fit & Framing Arrangements

- Language added to Article C6.7.4.2 to discuss beneficial framing arrangements in skewed and curved I-girder bridges to alleviate detrimental transverse stiffness effects.
- Revision made to recommended offset of first intermediate cross-frame placed normal to the girders adjacent to a skewed support.



Skewed and Curved I-Girder Bridge Fit & Framing Arrangements



- Framing of a normal intermediate cross-frame into or near a bearing location along a skewed support line is strongly discouraged unless the cross-frame diagonals are omitted.
- At skewed interior piers & abutments, place cross-frames along the skewed bearing line, and locate intermediate cross-frames greater than or equal to the recommended minimum offset from the bearing lines.
- For curved I-girder bridges, provide contiguous intermediate cross-frame lines within the span in combination with the recommended offset at skewed bearing lines.

Primary vs. Secondary Members, Charpy Requirements, FCMs & SRMs

Description of Specification Revisions:

Fracture-Critical Member (FCM)—~~Component in tension whose failure is expected to result in the collapse of the bridge or the inability of the bridge to perform its function.~~ <u>A steel primary member or portion thereof subject to tension whose failure would probably cause a portion of or the entire bridge to collapse.</u>

Primary Member—~~A member designed to carry the internal forces determined from an analysis.~~ <u>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".</u>

Secondary Member—~~A member in which stress is not normally evaluated in the analysis.~~ <u>A steel member or component that does not transmit gravity loads through a necessary as-designed load path.</u>

Primary vs. Secondary Members, Charpy Requirements, FCMs & SRMs

– A new Article 6.6.2.1 entitled ‘Member or Component Designations & Charpy V-Notch Testing Requirements’ is introduced.

- A new Table 6.6.2.1-1 is provided designating various members or components as primary or secondary.
- Primary members subject to a net tensile stress under Strength I are to be designated on the contract plans.
- Charpy V-notch testing is required for primary members subject to a net tensile stress under Strength I, *except for diaphragm and cross-frame members and mechanically fastened or welded cross-frame gusset plates in horizontally curved bridges*.

Member or Component Description	Member or Component Designation
Girders, beams, stringers, floorbeams, bent caps, bulkheads, and straddle beams	Primary
Truss chords, diagonals, verticals, and portal and sway bracing members	Primary
Arch ribs and built-up or welded tie girders	Primary
Rigid frames	Primary
Gusset plates and splice plates in trusses, arch ribs, tie girders, and rigid frames	Primary
Splice plates and cover plates in girders, beams, stringers, floorbeams, bent caps, and straddle beams	Primary
Bracing members supporting arch ribs	Primary
Permanent bottom-flange lateral bracing members and mechanically fastened or welded bottom-flange lateral connection plates in straight and horizontally curved bridges	Primary
Diaphragm, cross-frame, and top-flange lateral bracing members, struts, and mechanically fastened or welded cross-frame gusset plates and top-flange lateral connection plates in straight and horizontally curved bridges	Secondary
Diaphragm and cross-frame members, and mechanically fastened or welded cross-frame gusset plates and bearing stiffeners at supports in bridges located in Seismic Zones 3 or 4	Primary
Bearings, filler plates, sole plates, and masonry plates	Secondary
Mechanically fastened or welded longitudinal web and flange stiffeners	Primary
Mechanically fastened or welded transverse intermediate web stiffeners, transverse flange stiffeners, bearing stiffeners, and vertical and lateral connection plates	Secondary

Primary vs. Secondary Members, Charpy Requirements, FCMs & SRMs

- A new Article 6.6.2.2 entitled 'Fracture-Critical Members (FCMs)' is introduced.
  - Contains all existing requirements related to FCMs.
  - *Primary* members that are FCMs are to be designated on the contract plans.
  - Members not subject to a net tensile stress under Strength I are not to be designated as FCMs.

System Redundant Member (SRM)—A steel primary member or portion thereof subject to tension for which the redundancy is not known by engineering judgment, but which is demonstrated to have redundancy through a refined analysis. SRMs must be identified and designated as such by the Engineer on the contract plans, and designated in the contract documents to be fabricated according to Clause 12 of the *AASHTO/AWS D1.5M/D1.5 Bridge Welding Code*. An SRM need not be subject to the hands-on in-service inspection protocol for a FCM as described in 23 CFR 650.

Significant Updates to Appear in the 9th Edition LRFD BDS

- Revisions to the L/85 Guideline
- Improvements to R_b for Longitudinally Stiffened Girders
- Revisions to Fatigue Detail Table 6.6.1.2.3-1
- Revisions to the Flexural Design Provisions for Tees & Double Angles
- Variable Web Depth Members
- Design Provisions for Noncomposite Box-Section Members

Revisions to the L/85 Guideline

- Description of Specification Revisions:
  - Moves the L/85 guideline from Article C6.10.3.4.1 (Deck Placement) to Article C6.10.2.2 (Girder Flange Proportioning).
  - Guideline intended to ensure that individual field sections are more stable and easier to handle during lifting, erection, and shipping.
  - Guideline should be used in conjunction with the flange proportioning limits in Article 6.10.2.2 to establish a minimum top-flange width for each unspliced girder field section.
  - Terms in the guideline will be redefined as follows (Eq. C6.10.2.2-1):

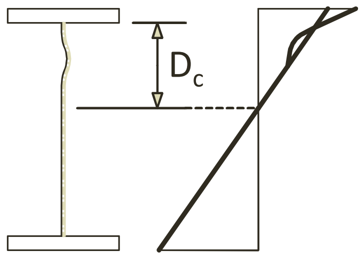
$$b_{tfs} \geq \frac{L_{fs}}{85}$$

  - The guideline is only to be applied to individual unspliced girder field sections for design.

Improvements to R_b for Longitudinally Stiffened Girders

- Description of Specification Revisions:
  - Improvements to the web load-shedding factor, R_b, for longitudinally stiffened steel girders.
  - Based on research by Lakshmi Subramanian and Don White at Georgia Tech – supported by AISI, AASHTO, FHWA, GDOT, and the MBMA.

Improvements to R_b for Longitudinally Stiffened Girders



- Maximum major-axis bending resistance:
  - Compression flange $F_{nc} = R_b R_h F_{yc}$
- R_b = 1 when
  - Section is composite in positive flexure, and $D/t_w \leq 150$
  - One or more longitudinal stiffeners are provided, and:

$$\frac{D}{t_w} \leq 0.95\sqrt{\frac{Ek}{F_{yc}}}$$

  - $2D_c/t_w \leq \lambda_{rw}$, where $\lambda_{rw} = 5.7\sqrt{E/F_{yc}}$ (i.e., web is nonslender)
- Otherwise: $R_b = 1.0 - \dfrac{a_{wc}}{1200 + 300a_{wc}}\left(\dfrac{2D_c}{t_w} - \lambda_{rw}\right) \leq 1.0$

Improvements to R_b for Longitudinally Stiffened Girders

- … when the web satisfies $2D_c / t_w \le \lambda_{rw}$, $R_b = 1.0$
- Otherwise: in lieu of a strain-compatibility analysis considering the web effective widths, for longitudinally-stiffened sections in which one or more continuous longitudinal stiffeners are provided that satisfy $d_s / D_c < 0.76$:

$$R_b = 1.07 - 0.12\frac{D_c}{D} - \frac{a_{wc}}{1200 + 300a_{wc}}\left[\frac{D}{t_w} - \lambda_{rwD}\right] \le 1.0$$

- For all other cases:

$$a_{wc} = \frac{2D_c t_w}{b_{fc} t_{fc}}$$

$$R_b = 1.0 - \frac{a_{wc}}{1200 + 300a_{wc}}\left(\frac{2D_c}{t_w} - \lambda_{rw}\right) \le 1.0$$

Improvements to R_b for Longitudinally Stiffened Girders

- Limit the transverse stiffener spacing in longitudinally stiffened webs to:

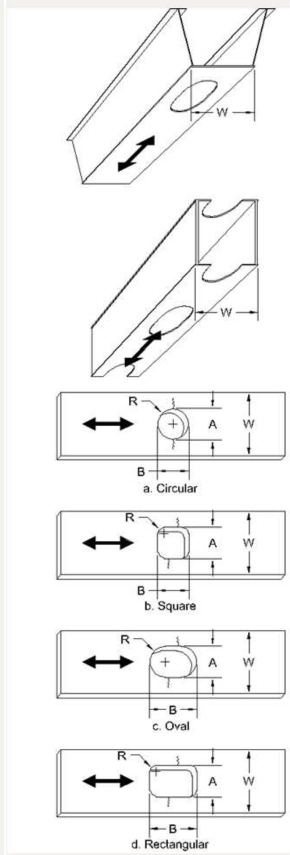
$$d_o \leq 2D$$

- Limit the curvature parameter in longitudinally stiffened webs to:

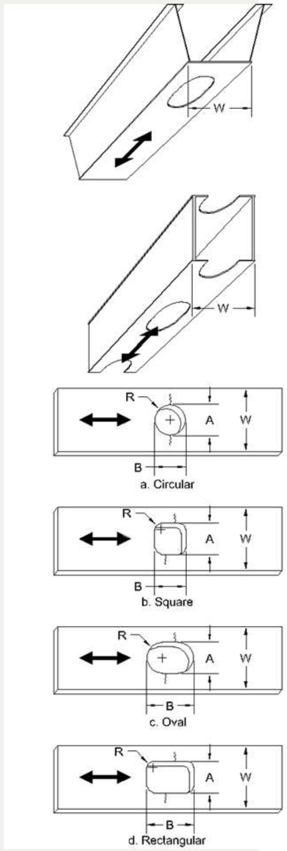
$$Z = \frac{0.95 d_o^2}{R t_w} \leq 12$$

- Indicate that longitudinal stiffeners should be included in calculating section properties of the member gross cross-section.

Revisions to Fatigue Detail Table 6.6.1.2.3-1

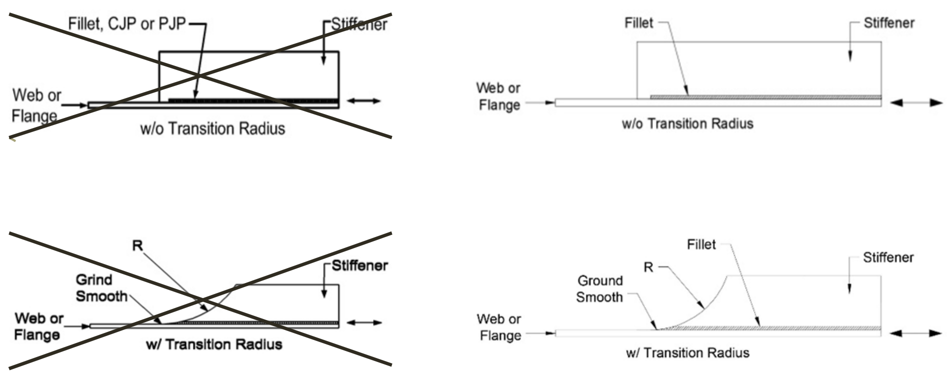
1.6 Base metal at the net section of manholes or hand holes made to the requirements of AASHTO/AWS D1.5, in which the width of the hole is at least 0.30 times the width of the plate (A ≥ 0.30W) (Bonachera Martin and Connor, 2017). The geometry of the hole shall be: a. circular; or b. square with corners filleted at a radius at least 0.10 times the width of the plate (R ≥ 0.10W); or c. oval (B > A), elongated parallel to the primary stress range; or	C	44 x 10^8	10	In the net section originating at the side of the hole	

Revisions to Fatigue Detail Table 6.6.1.2.3-1

d. rectangular ($B > A$), elongated parallel to the primary stress range, with corners filleted at a radius at least 0.10 times the width of the plate ($R \geq 0.10W$). All holes shall be centered on the plate under consideration, and all stresses shall be computed on the net section. (Note: Condition 1.5 shall apply for all holes in cross-sections in which other smaller open holes or holes with nonpretensioned fasteners are located anywhere within the net section of the larger hole, and minimum edge distance requirements specified in Article 6.13.2.6.6 are satisfied for the smaller holes.)	C	44×10^8	10	In the net section originating at the side of the hole	

Revisions to Fatigue Detail Table 6.6.1.2.3-1

- The term “flame-cut” will be changed to the more general term “thermal-cut” in the descriptions for Conditions 1.1, 1.2, and 8.7 in Table 6.6.1.2.3-1.
- The sketches in Condition 4.3 in Table 6.6.1.2.3-1 will be revised as follows:



Revisions to the Flexural Design Provisions for Tees & Double Angles

Description of Specification Revisions:

- Revisions are made to Articles 6.12.2.2.4 and C6.12.2.2.4 for determining the flexural resistance of tees and double angles loaded in the plane of symmetry in order to bring the provisions up-to-date with the latest provisions in AISC (2016).
  - Prior editions of the AISC Specification did not distinguish between tees and double angles and as a result, there were instances when double angles would appear to have less strength than two single angles. This concern is now addressed by providing separate provisions for tees and double angles.
  - In those cases where double angles should have the same strength as two single angles, the revised provisions make use of the equations for single angles, as applicable, given in Section F10 of AISC (2016).

Revisions to the Flexural Design Provisions for Tees & Double Angles

- In addition, a new linear transition equation from M_p to M_y is introduced for the limit state of lateral-torsional buckling when the stem of the member is in tension; that is, when the flange is subject to compression. Previous specifications transitioned abruptly from the full plastic moment to the elastic buckling range.

For ~~lateral torsional buckling~~ <u>tee stems and double angle web legs subject to tension</u>, the nominal flexural resistance <u>based on lateral-torsional buckling</u> shall be taken as:

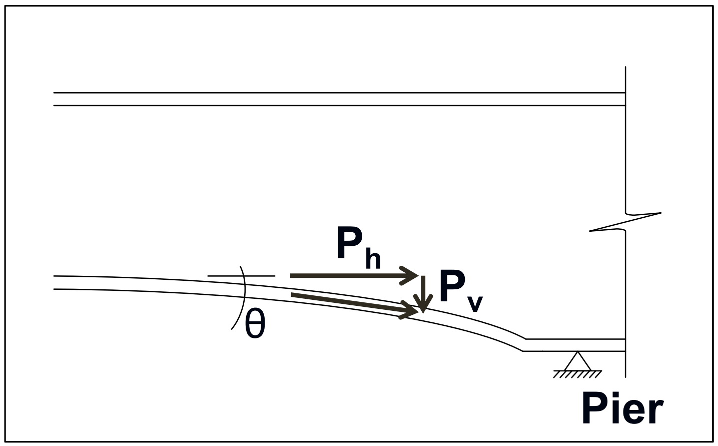
- <u>If $L_b \le L_p$, then lateral-torsional buckling shall not apply.</u>
- <u>If $L_p < L_b \le L_r$, then:</u>

$$M_n = M_p - \left(M_p - M_y\right)\left(\frac{L_b - L_p}{L_r - L_p}\right) \qquad \text{(6.12.2.2.4c-1)}$$

- <u>If $L_b > L_r$, then:</u>

$$M_n = M_{cr} \qquad \text{(6.12.2.2.4c-2)}$$

Variable Web Depth Members



Horizontal component of force in flange:

$$P_h = M\frac{A_f}{S_x}$$

Normal stress in inclined flange:

$$f_n = \frac{P_h}{A_f \cos\theta}$$

Vertical component of force in flange:

$$P_v = P_h \tan\theta$$

Variable Web Depth Members

- A provision in Article 6.10.1.4 on Variable Web Depth Members will be revised as follows:

6.10.1.4—Variable Web Depth Members

At points where the bottom flange becomes horizontal, ~~the transfer of the vertical component of the flange force back into the web shall be considered.~~ <u>full- or partial-depth transverse stiffening of the web shall be provided, unless the provisions of Article D6.5.2 are satisfied for the factored vertical component of the inclined flange force using a length of bearing *N* equal to zero.</u>

Variable Web Depth Members

D6.5.2—Web Local Yielding

Webs subject to compressive or tensile concentrated loads shall satisfy:

$$R_u \leq \phi_b R_n \quad \text{(D6.5.2-1)}$$

in which:

R_n = nominal resistance to the concentrated loading (kip)

- For interior-pier reactions and for concentrated loads applied at a distance from the end of the member that is greater than d:

$$R_n = (5k + N) F_{yw} t_w \quad \text{(D6.5.2-2)}$$

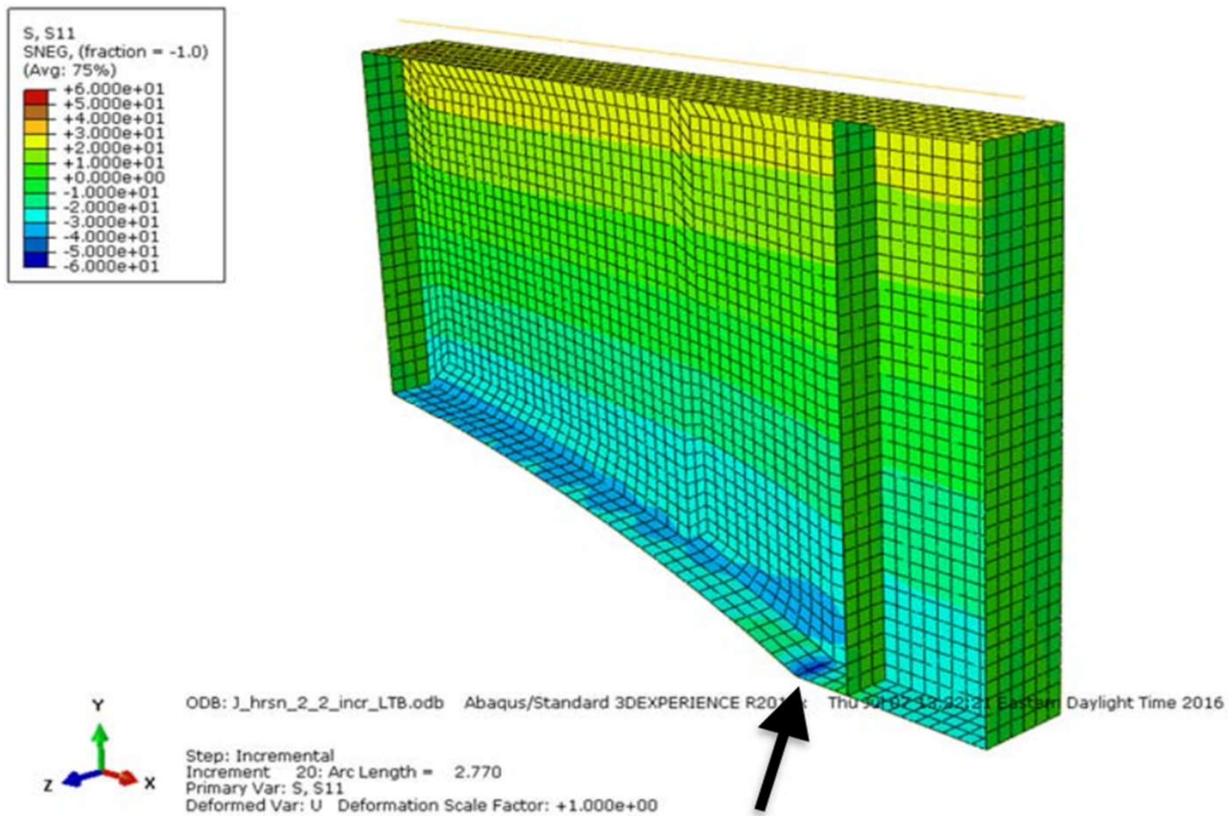
- Otherwise:

$$R_n = (2.5k + N) F_{yw} t_w \quad \text{(D6.5.2-3)}$$

where:

ϕ_b = resistance factor for bearing specified in Article 6.5.4.2
d = depth of the steel section (in.)
k = distance from the outer face of the flange resisting the concentrated load or bearing reaction to the web toe of the fillet (in.)
N = length of bearing (in.). N shall be greater than or equal to k at end bearing locations.
R_u = factored concentrated load or bearing reaction (kip)

Variable Web Depth Members



Design Provisions for Noncomposite Box-Section Members

- Description of Specification Revisions:
  - Implementation of a more general and consistent approach for the LRFD design of unstiffened and stiffened compression elements in all noncomposite box sections (i.e., box sections utilized in trusses, arches, frames, straddle beams, etc.) subject to uniform stress (compression) or nonuniform stress (e.g. compression plus bending or compression plus bending plus shear and/or torsion, etc.)
  - Based on research being conducted under FHWA IDIQ Task Order 5011 managed by HDR Engineering
  - Project Team:
    - Don White, Georgia Tech (Technical PI)
    - Ajinkya Lokhande, Georgia Tech
    - John Yadlosky, HDR Engineering
    - Charles King, COWI
    - Mike Grubb, M.A. Grubb & Associates
    - Tony Ream, HDR Engineering
    - Frank Russo, Michael Baker International, LLC

Design Provisions for Noncomposite Box-Section Members

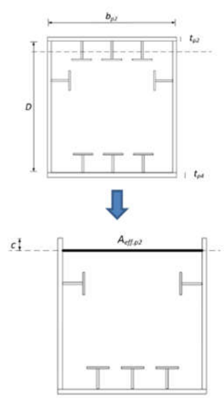
- Benefits:
  - Unstiffened and longitudinally stiffened noncomposite *rectangular* box-section members
  - Built-up welded boxes, bolted boxes, and square and rectangular HSS
  - Singly- and doubly-symmetric rectangular sections
  - Homogeneous and hybrid sections
  - All ranges of web and flange plate slenderness
  - Use of an effective compression flange width in determining cross-section properties for boxes with noncompact and slender compression flanges (rely on post-buckling resistance)
  - No theoretical shear buckling or plate local buckling permitted at the fatigue and service limit states, and for constructibility
  - Use of a web plastification factor for sections having noncompact or compact webs (allows flexural resistances > M_{ye})

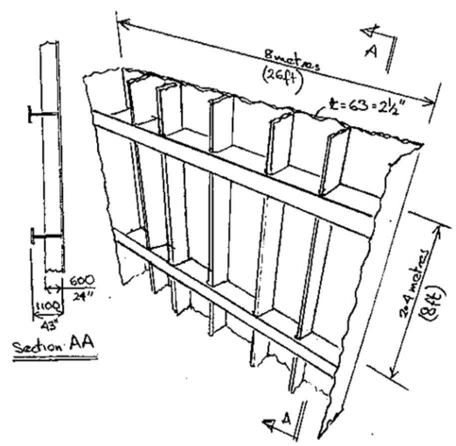
Design Provisions for Noncomposite Box-Section Members

- Benefits (cont.):
  - No need to check elastic LTB; accuracy with respect to the limit state of inelastic LTB is significantly improved
  - More efficient b/t limits for solid web arches
  - Eliminates reliance on LFD Truss Guide Specifications
  - Handles interaction of all force effects, including torsion
  - Provides improved provisions for longitudinally stiffened flanges (new Appendix E6):
    - Provide same set of equations for any number of stiffeners, transversely stiffened or not
    - Take advantage of longitudinal stiffener, transverse stiffener and stiffened plate contributions to compression capacity
    - Allows designer to easily determine from equation components if longitudinally and/or transverse stiffening is effective
  - Obtain more accurate and sufficient ratings for existing structures outside the slenderness limits of the current Specifications, or with inadequate stiffeners

Design Provisions for Noncomposite Box-Section Members

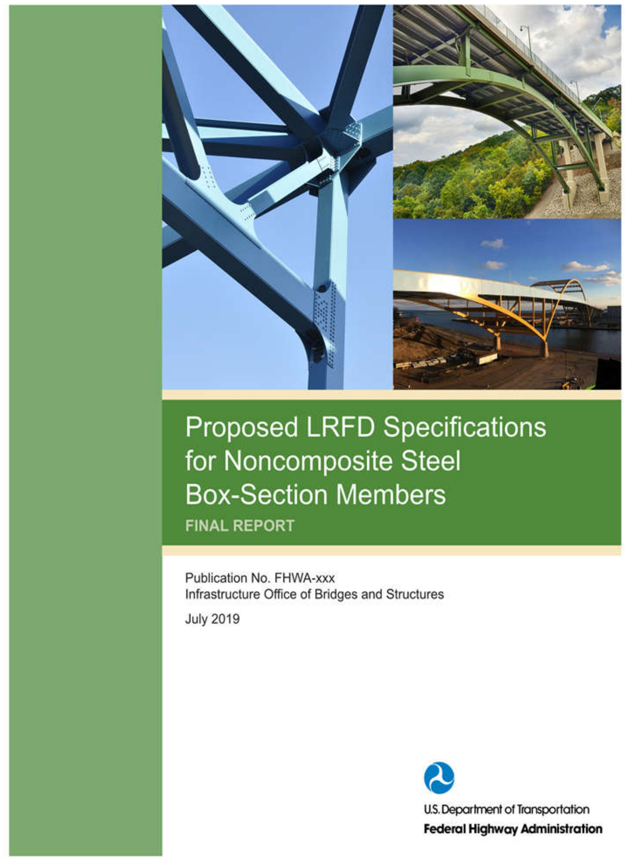
- Benefits (cont.):
  - Stiffened slender boxes have the potential to reduce weight for large structures, such as steel tower legs for cable stayed bridges
  - Specifications are more streamlined and user-friendly
  - Similar, but better prediction results relative to current AASHTO & AISC, where the current AISC & AASHTO are actually applicable … and similar, but better, predictions compared to Eurocode, BS5400 (pre Eurocode), and Wolchuk & Mayrbaurl (1980)

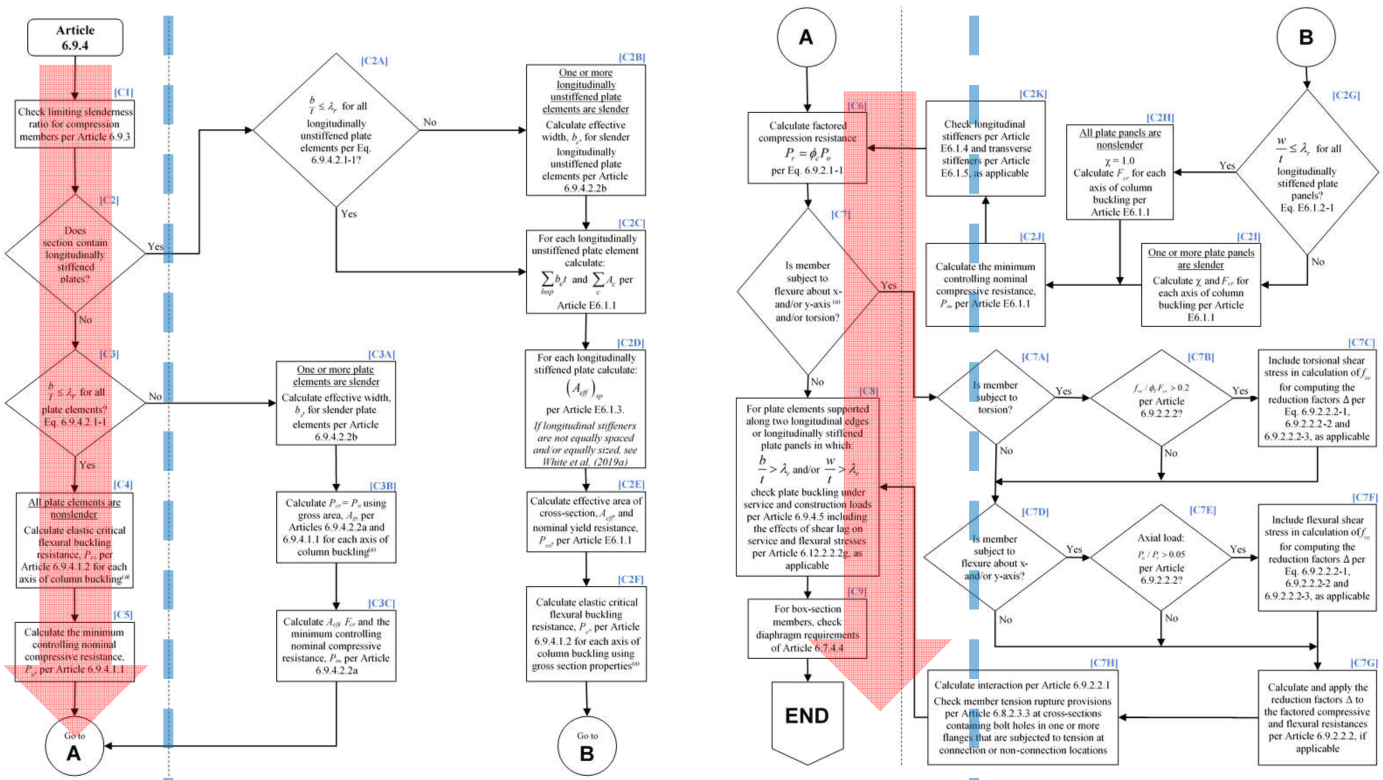




Design Provisions for Noncomposite Box-Section Members

- **"Proposed LRFD Specifications for Noncomposite Steel Box-Section Members"**
  - FHWA-HIF-19-063 | July 2019
  - (NCHRP 20-07/415)
- **Expanded Commentary**
- **Additional provisions for specialized situations**
- **3 Examples:**
  - Longitudinally Unstiffened Truss End Post
  - Longitudinally Stiffened/Slender Tie Girder
  - Longitudinally Stiffener Arch Rib
- **2 Flowcharts coordinated with Examples**
  - Compression & Flexural Resistance





Article 6.9.4
[C1]
Check limiting slenderness ratio for compression members per Article 6.9.3
[C2]
Does section contain longitudinally stiffened plates?
Yes
No
[C2A]
$\frac{b}{t} \le \lambda_r$ for all longitudinally unstiffened plate elements per Eq. 6.9.4.2.1-1?
No
Yes
[C2B]
One or more longitudinally unstiffened plate elements are slender
Calculate effective width, b_e, for slender longitudinally unstiffened plate elements per Article 6.9.4.2.2b
[C2C]
For each longitudinally unstiffened plate element calculate:
$\sum_{lusp} b_e t$ and $\sum_c A_c$ per Article E6.1.1
[C2D]
For each longitudinally stiffened plate calculate:
$\left(A_{eff}\right)_{sp}$
per Article E6.1.3.
If longitudinal stiffeners are not equally spaced and/or equally sized, see White et al. (2019a)
[C2E]
Calculate effective area of cross-section, A_{eff}, and nominal yield resistance, P_{os}, per Article E6.1.1
[C2F]
Calculate elastic critical flexural buckling resistance, P_e, per Article 6.9.4.1.2 for each axis of column buckling using gross section properties(a)
Go to B
[C3]
$\frac{b}{t} \le \lambda_r$ for all plate elements? Eq. 6.9.4.2.1-1
No
Yes
[C3A]
One or more plate elements are slender
Calculate effective width, b_e, for slender plate elements per Article 6.9.4.2.2b
[C3B]
Calculate $P_{cr} = P_n$ using gross area, A_g, per Articles 6.9.4.2.2a and 6.9.4.1.1 for each axis of column buckling(a)
[C3C]
Calculate A_{eff}, F_{cr} and the minimum controlling nominal compressive resistance, P_n, per Article 6.9.4.2.2a
[C4]
All plate elements are nonslender
Calculate elastic critical flexural buckling resistance, P_e, per Article 6.9.4.1.2 for each axis of column buckling(a)
[C5]
Calculate the minimum controlling nominal compressive resistance, P_n, per Article 6.9.4.1.1
Go to A
A
[C6]
Calculate factored compression resistance
$P_r = \phi_c P_n$
per Eq. 6.9.2.1-1
[C7]
Is member subject to flexure about x- and/or y-axis (a) and/or torsion?
Yes
No
[C8]
For plate elements supported along two longitudinal edges or longitudinally stiffened plate panels in which:
$\frac{b}{t} > \lambda_r$ and/or $\frac{w}{t} > \lambda_r$
check plate buckling under service and construction loads per Article 6.9.4.5 including the effects of shear lag on service and flexural stresses per Article 6.12.2.2.2g, as applicable
[C9]
For box-section members, check diaphragm requirements of Article 6.7.4.4
END
B
[C2G]
$\frac{w}{t} \le \lambda_r$ for all longitudinally stiffened plate panels? Eq. E6.1.2-1
Yes
No
[C2H]
All plate panels are nonslender
$\chi = 1.0$
Calculate F_{cr} for each axis of column buckling per Article E6.1.1
[C2I]
One or more plate panels are slender
Calculate χ and F_{cr} for each axis of column buckling per Article E6.1.1
[C2J]
Calculate the minimum controlling nominal compressive resistance, P_n, per Article E6.1.1
[C2K]
Check longitudinal stiffeners per Article E6.1.4 and transverse stiffeners per Article E6.1.5, as applicable
[C7A]
Is member subject to torsion?
Yes
No
[C7B]
$f_{ve} / \phi_v F_{cv} > 0.2$ per Article 6.9.2.2.2?
Yes
No
[C7C]
Include torsional shear stress in calculation of f_{ve} for computing the reduction factors Δ per Eq. 6.9.2.2.2-1, 6.9.2.2.2-2 and 6.9.2.2.2-3, as applicable
[C7D]
Is member subject to flexure about x- and/or y-axis?
Yes
No
[C7E]
Axial load: $P_u / P_r > 0.05$ per Article 6.9.2.2.2?
Yes
No
[C7F]
Include flexural shear stress in calculation of f_{ve} for computing the reduction factors Δ per Eq. 6.9.2.2.2-1, 6.9.2.2.2-2 and 6.9.2.2.2-3, as applicable
[C7G]
Calculate and apply the reduction factors Δ to the factored compressive and flexural resistances per Article 6.9.2.2.2, if applicable
[C7H]
Calculate interaction per Article 6.9.2.2.1
Check member tension rupture provisions per Article 6.8.2.3.3 at cross-sections containing bolt holes in one or more flanges that are subjected to tension at connection or non-connection locations

Questions?