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15th Edition Steel Construction Manual
Part 16 - What to Expect:

A Review of ANSI/AISC 360-16

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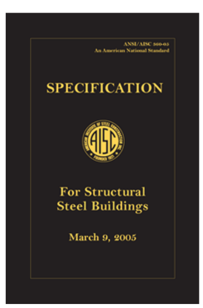
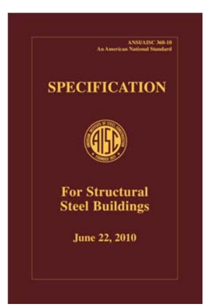
2016 Specification, where did we start?

- AISC 360-05, published March 9, 2005
- AISC 360-10, published June 22, 2010
- AISC 360-16, published July 7, 2016





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2016 Specification, what can you expect?

- AISC's continued commitment to a single specification for all types of elements.
- Continued availability of provisions for LRFD and ASD (*a unified specification*)

$$R_u \leq \phi R_n \quad \text{and} \quad R_a \leq R_n / \Omega$$

- Some expansion but overall, minimal change.

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What was the goal for 2016?

- To again produce a specification that reflects minimal change from previous editions.
- Implement only essential changes.
- Continue to coordinate with all AISC standards
- Integrate with non AISC standards such as ASCE 7, IBC, and ACI.
- Reference appropriate other newly approved standards.

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2016 AISC Specification

- Basic organization has not changed
  - Still 14 Chapters, A through N
  - Still 8 Appendices, 1 through 8
- Within some chapters, provisions have been reorganized
- Throughout there have been editorial changes that will not be discussed here

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Chapter A General Provisions

- A3. Material
  - Tubing and pipe are now combined under Section A3.1a(b) Hollow Structural Sections (HSS)
    - ASTM A1065, 50 ksi welded HSS added
    - ASTM A1085, new HSS standard with tighter tolerances added
  - ASTM A1066, HSLA plate produced by thermo-mechanical controlled process added
    - Grades 50 to 80

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Chapter A General Provisions

- A3.3 Bolts, washers and nuts
  - ASTM F3125 added. It includes the former A325 and A490 type standards as grades.
  - Thus, ASTM A325, A490, F1852, and F2280 no longer exist as separate standards.
  - ASTM F3111 and F3040, 200 ksi hex head and twist-off bolts are added as Group C. These are proprietary products

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Chapter B Design Requirements

- B3. Design Basis
  - This section has been completely reorganized
  - It provides the charging language for the remainder of the Specification in this new order:
    1. Design by LRFD
    2. Design by ASD
    3. Required Strength
    4. Design of Connections and Supports
    5. Design of Diaphragms and Collectors

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Chapter B Design Requirements

- B3. Design Basis
  6. Design of Anchorages to Concrete
  7. Design for Stability
  8. Design for Serviceability
  9. Design for Structural Integrity – new provisions
  10. Design for Ponding – revised provisions
  11. Design for Fatigue
  12. Design for Fire Conditions
  13. Design for Corrosion Effects

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Chapter B Design Requirements

- B3.9 Design for Structural Integrity
  – When required by applicable building code
    (a) Column splice tensile strength
    (b) Beam end connection tension strength
    (c) Bracing connection tensile strength

Strength requirements for structural integrity evaluated independently of other strength requirements.

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Chapter B Design Requirements

- B3.10 Design for Ponding
  - Requirement regarding minimum roof slope to avoid ponding analysis is removed.
  - Now requires that stability for ponding be checked unless roof surface is configured to prevent the accumulation of water.
  - Appendix 2 is referenced and there the assumptions for use of the two approaches are defined.

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Chapter B Design Requirements

- B4.2 Design Wall Thickness for HSS
  - Two new material specifications, ASTM A1065 and A1085.
    - Both permit use of nominal thickness as design wall thickness
    - A1085 – an HSS standard with tighter tolerances than A500 etc.
    - A1065 – 50 ksi plate material produced to a standard with typical plate tolerances.

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Chapter B Design Requirements

- B4.2 Design Wall Thickness for HSS

  User Note: A pipe can be designed using the provisions of the Specification for round HSS sections as long as the pipe conforms to ASTM A53 Class B and the appropriate limitations of the Specification are used.

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Chapter C Design for Stability

- C2.2 Consideration of Initial System Imperfections
  - System Imperfections
    - Imperfections in location of points of intersection of members (column out-of-plumbness)
  - Member Imperfections
    - User note now makes it clear that these are already taken into account in the column equations from Chapter E. (column out-of-straightness)

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Chapter C Design for Stability

- C2.3(b) Adjustments to Stiffness
  - Flexural stiffness reduction factor, τ_b, now defined for composite members (in Chapter I)
  - Also redefined for members with slender elements

$$\tau_b = 4\left(\alpha P_r / P_{ns}\right)\left[1-\left(\alpha P_r / P_{ns}\right)\right] \quad \text{(C2-2b)}$$

    - For nonslender element sections $P_{ns} = F_y A_g$
    - For slender element sections $P_{ns} = F_y A_e$

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Chapter C Design for Stability

- *K*-factor
  - First introduced in the 1963 Specification
  - Set equal to 1.0 when using the direct analysis method of Chapter C (since 2005)
  - For 2016, *KL,* the effective length, has been replaced with L_c. This makes the designation of effective length simpler since in some instances, such as for torsion, the traditional definition of *K* is not helpful.
  - This has been implemented throughout the 2016 Specification

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Chapter D Tension

- The only change in this chapter has to do with shear lag for longitudinally welded connections to tension members.
- Case 2 no longer applies to welded connections unless they use transverse welds in combination with longitudinal welds.

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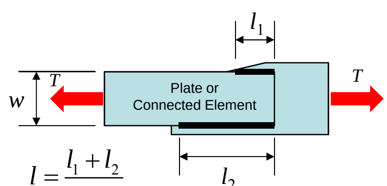
Chapter D Tension

- Thus, Case 4 has been expanded to include shapes as well as plates and the shear lag factor equation revised.

  Case 4: Plates, angles, channels with welds at heels, tees, and W-shapes with connected elements, where the tension load is transmitted by longitudinal welds only

$$U = \frac{3l^2}{3l^2 + w^2}\left(1 - \frac{\bar{x}}{l}\right)$$



$$l = \frac{l_1 + l_2}{2}$$

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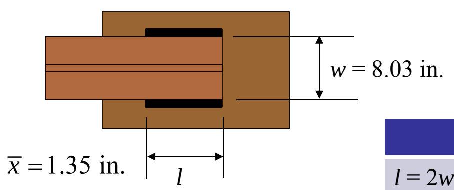
Example 1

- Determine the shear lag factor for a WT7x24 welded to a plate with longitudinal welds at the flange tips.

$$U = \frac{3l^2}{3l^2 + w^2}\left(1 - \frac{\bar{x}}{l}\right)$$



$$A_e = UA_n$$

	U_{2010}	U_{2016}
$l = 2w = 16.1$	0.916	0.923(0.916)=0.845
$l = 1.5w = 12.0$	0.888	0.871(0.888)=0.773
$l = w = 8.03$	0.832	0.750(0.832)=0.624

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Chapter E Compression

- E4. Torsional and Flexural-Torsional Buckling of Single Angles and Members without Slender Elements
  - Consider what to do when twisting is not about the shear center.
  - Deleted the special case for double angles and tees.
  - Clarified that this section applies to single angles with

$$b/t > 0.71\sqrt{E/F_y}$$

(This had been *b*/*t* > 20 in 2010)

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Chapter E Compression

- E5. Single Angle Compression Members
  - Clarified that flexural-torsional buckling need not be considered for $b/t \le 0.71\sqrt{E/F_y}$
- E6. Built-up members
  - Clarified need for Class A or B faying surfaces in some situations.
- E7. Members with Slender Elements
  - Completely revised the approach and made it similar to AISI approach.

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Chapter E Compression

- E7. Members with Slender Elements
  - Stiffened and unstiffened elements treated similarly (same effective width equation)
  - The critical stress is the same, regardless of element slenderness (E3-2, E3-3)
  - Slender element comes into play through effective area

$$P_n = F_{cr} A_e \qquad \text{(E7-1)}$$

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Chapter E Compression

- E7. Members with Slender Elements
  - when

$$\lambda \leq \lambda_r \sqrt{\frac{F_y}{F_{cr}}}$$

$$b_e = b \qquad \text{(E7-2)}$$

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Chapter E Compression

- E7. Members with Slender Elements
  - when

$$\lambda > \lambda_r \sqrt{\frac{F_y}{F_{cr}}}$$

$$b_e = b\left(1 - c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} \qquad \text{(E7-3)}$$

$$F_{el} = \left(c_2 \frac{\lambda_r}{\lambda}\right)^2 F_y \qquad \text{(E7-4)}$$

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Chapter E Compression

$$c_2 = \frac{1-\sqrt{1-4c_1}}{2c_1}$$

Table E7-1

Effective Width Imperfection Adjustment Factor, c_1 and c_2 Factor.

Case	Slender Element	c_1	c_2
(a)	Stiffened elements except walls of square and rectangular HSS	0.18	1.31
(b)	Walls of square and rectangular HSS	0.20	1.38
(c)	All other elements	0.22	1.49

$$\lambda_r = c_3\sqrt{\frac{k_c E}{F_y}}$$

There are 6 distinct values for c_3, found in Table B4.1a

$k_c = 1$ except for flanges of built-up I-shaped sections

Round HSS still treated differently

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Chapter E Compression

- E7. Members with Slender Elements

$$b_e = b\left(1-c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} \qquad \text{(E7-3)}$$

which, for webs of I-shaped members, ends up being

$$b_e = 1.95t\sqrt{\frac{E}{F_{cr}}}\left[1-\frac{0.351}{(b/t)}\sqrt{\frac{E}{F_{cr}}}\right]$$

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Chapter E Compression

- E7. Members with Slender Elements

$$b_e = 1.95t\sqrt{\frac{E}{F_{cr}}}\left[1-\frac{0.351}{(b/t)}\sqrt{\frac{E}{F_{cr}}}\right]$$

which for 2010 was

SAME

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1-\frac{0.34}{(b/t)}\sqrt{\frac{E}{f}}\right]$$

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Chapter E Compression

- E7. Members with Slender Elements

2010

2016

$$A_e = A_g - (b-b_e)t$$

$$Q = \frac{A_e}{A_g}$$

$$F_{cr} = \left(0.658^{\frac{QF_y}{F_e}}\right)QF_y$$

$$P_n = F_{cr}A_g$$

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right)F_y$$

$$P_n = F_{cr}A_e$$

$$\boxed{A_e = QA_g}$$

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Example 2

- Determine the compressive strength of a slender flange I-shape. $L_c = KL = 20\text{ ft}$

Flange: 24 x 0.5 in.
Web: 24 x 0.75 in.

$r_y = 5.24\text{ in.}$

$$k_c = \frac{4}{\sqrt{h/t_w}} = 0.707$$

$$\lambda_{rf} = 0.64\sqrt{k_c E/F_y} = 13.0 < b_f/2t_f = 24$$

Thus, the flange is slender

For 2010

$$\frac{b}{t} = 24 > 1.17\sqrt{k_c E/F_y} = 23.7$$

$$Q_s = \frac{0.90Ek_c}{F_y(b/t)^2} = 0.641$$

$$\frac{L_c}{r} = \frac{20(12)}{5.24} = 45.8$$

$$F_e = \frac{\pi^2 E}{(L_c/r)^2} = 136\text{ ksi}$$

$$\frac{QF_y}{F_e} = \frac{0.641(50)}{136} = 0.236 < 2.25$$

$$F_{cr} = 0.641\left(0.658^{(0.236)}\right)(50) = 29.0\text{ ksi}$$

$$P_n = 29.0(42.0) = 1220\text{ kips}$$

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Example 2

- For 2016, start with F_{cr}

$$\boxed{\begin{aligned}\frac{L_c}{r} &= \frac{20(12)}{5.24} = 45.8\\ F_e &= \frac{\pi^2 E}{(L_c/r)^2} = 136\text{ ksi}\end{aligned}}$$

$$\frac{F_y}{F_e} = \frac{50}{136} = 0.368 < 2.25$$

$$F_{cr} = 0.658^{(0.368)}(50) = 42.9\text{ ksi}$$

$$\lambda_{rf} = 13.0$$

$$b_f/2t_f > \lambda_{rf}\sqrt{F_y/F_{cr}} = 13.0\sqrt{50/42.9} = 14.0$$

Thus, the flange is slender

$$b_e = b\left(1 - c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}}$$

$$= 12\left(1 - 0.22\sqrt{\frac{32.6}{42.9}}\right)\sqrt{\frac{32.6}{42.9}} = 8.45$$

$$A_e = 24(0.75) + 24(0.5) + 2(8.45)(0.5) = 38.5\text{ in.}^2$$

$$P_n = 42.9(38.5) = 1650\text{ kips}$$

$$Q = \frac{A_e}{A_g} = \frac{38.5}{42.0} = 0.917$$

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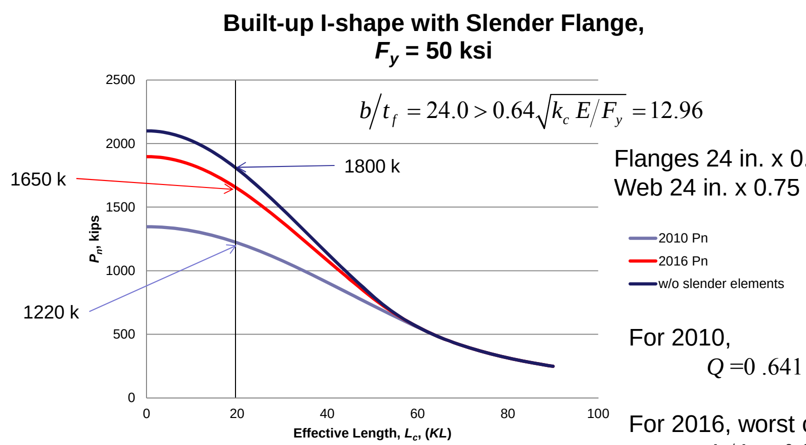
Chapter E Compression



Flanges 24 in. x 0.5 in.
Web 24 in. x 0.75 in.

For 2010,
$Q = 0.641$

For 2016, worst case
$A_e/A_g = 0.904$

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Chapter F Flexure

- F1. General Provisions
  - Clarify C_b for cantilevers
- F4. and F5. Noncompact and slender webs
  - Revised effective radius of gyration for LTB
- F7. HSS and Box Sections
  - Clarified here and throughout that box sections were doubly symmetric and treated like rectangular HSS
  - For box sections added slender web and lateral-torsional buckling

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Chapter F Flexure

- F9. Tees and Double Angles
  - Revised LTB provisions
  - Revised stem local buckling
  - Cleaned-up and reorganized section
- F10. Single Angles
  - Reduced conservatism for LTB by making a more realistic assumption for b/t
- F13.1 Holes in tension flanges
  - Clarified use of minimum section modulus

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Chapter F Flexure

- F9. Tees and Double Angles – LTB

2010

L_p and L_r not used

$$M_n = M_{cr} = \frac{\pi\sqrt{EI_y GJ}}{L_b}\left(B + \sqrt{1+B^2}\right)$$

$$B = \pm 2.3\left(\frac{d}{L_b}\right)\sqrt{\frac{I_y}{J}}$$

Stem in tension, +*B*
Stem in compression, -*B*

Same equations

2016

Stems in tension

$L_p < L_b \le L_r$ In this range, 2016 more conservative

$$M_n = M_p - \left(M_p - M_y\right)\left[\frac{L_b - L_p}{L_r - L_p}\right]$$

$L_b > L_r$

$$M_n = M_{cr} = \frac{1.95E}{L_b}\sqrt{I_y J}\left[B + \sqrt{1+B^2}\right]$$

Stems and web legs in compression
Tees and double angles treated a bit differently

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Chapter F Flexure

- F9. Tees and Double Angles – Stem LB

2010

$$0.84\sqrt{\frac{E}{F_y}} < \frac{d}{t_w} \le 1.03\sqrt{\frac{E}{F_y}}$$

$$F_{cr} = \left[2.55 - 1.84\frac{d}{t_w}\sqrt{\frac{F_y}{E}}\right]F_y$$

$$\frac{d}{t_w} > 1.03\sqrt{\frac{E}{F_y}}$$

$$F_{cr} = \frac{0.69E}{\left(\frac{d}{t_w}\right)^2}$$

2016

$$0.84\sqrt{\frac{E}{F_y}} < \frac{d}{t_w} \le 1.52\sqrt{\frac{E}{F_y}}$$

$$F_{cr} = \left[1.43 - 0.515\frac{d}{t_w}\sqrt{\frac{F_y}{E}}\right]F_y$$

$$\frac{d}{t_w} > 1.52\sqrt{\frac{E}{F_y}}$$

$$F_{cr} = \frac{1.52E}{\left(\frac{d}{t_w}\right)^2}$$

Double angles treated like 2 single angles

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Example 3

- Determine the nominal moment strength for the limit state of stem local buckling for a WT12x34 $d/t_w = 28.7, \quad S_x = 15.6 \text{ in.}^3$

2010 Limits

$$\lambda_p = 0.84\sqrt{E/F_y} = 20.2$$

$$\lambda_r = 1.03\sqrt{E/F_y} = 24.8$$

$$d/t_w = 28.7 > \lambda_r$$

$$F_{cr} = \frac{0.69E}{(d/t_w)^2} = 24.3 \text{ ksi}$$

$$M_n = F_{cr}S_x = 24.3(15.6) = 379 \text{ in.-kips}$$

2016 Limits

$$\lambda_p = 0.84\sqrt{E/F_y} = 20.2$$

$$\lambda_r = 1.52\sqrt{E/F_y} = 36.6$$

$$\lambda_p < d/t_w = 28.7 < \lambda_r$$

$$F_{cr} = \left(1.43 - 0.515\frac{d}{t_w}\sqrt{\frac{F_y}{E}}\right)F_y = 40.8 \text{ ksi}$$

$$M_n = F_{cr}S_x = 40.8(15.6) = 636 \text{ in.-kips}$$

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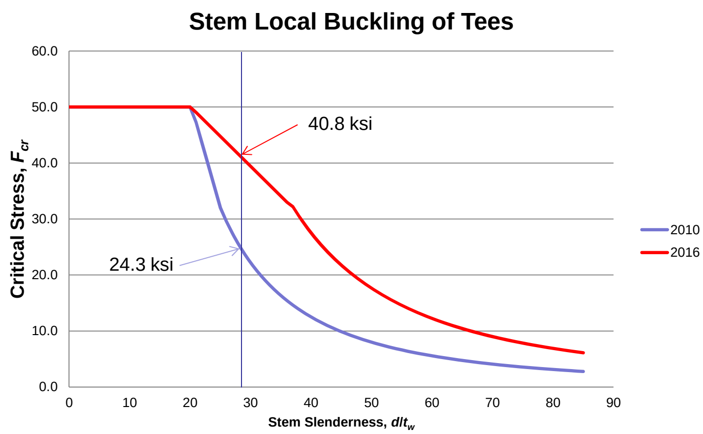
Chapter F Flexure



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Chapter G Shear

- G2. I-shaped Members and Channels
  - G2.1 Shear Strength of Webs without Tension Field Action
    - Increased strength by accounting for some post-buckling strength of web
  - G2.2 Shear Strength of Interior Web Panels with $a/h \leq 3$ Considering Tension Field Action
    - Expanded tension field action beyond the limits found in 2010.

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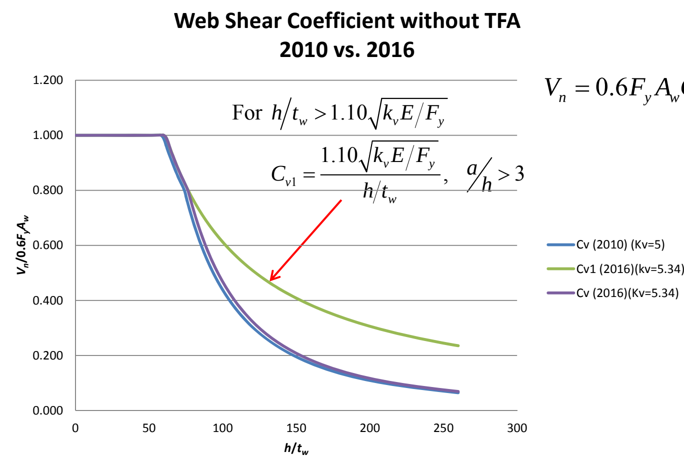
Chapter G Shear

$$V_n = 0.6F_yA_wC_{v1}$$



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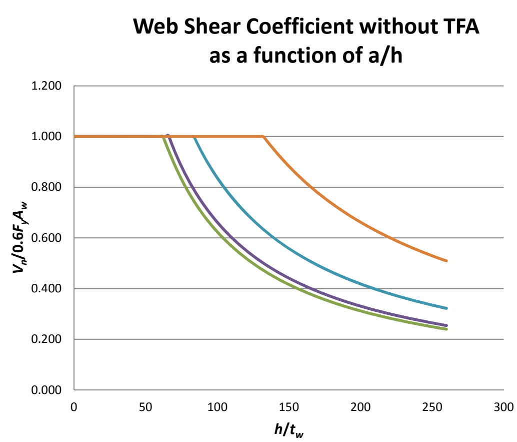
Chapter G Shear



$$V_n = 0.6F_yA_wC_{v1}$$

$$C_{v1} = \frac{1.10\sqrt{k_vE/F_y}}{h/t_w}$$

$$k_v = 5 + \frac{5}{(a/h)^2}$$

$$= 5.34 \text{ when } a/h > 3.0$$

Cv1(3.0)
Cv1(2.0)
Cv1(1.0)
Cv1(0.5)

For stiffened panels with *a*/*h* = ( )

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Chapter G Shear

- Tension field action is extended beyond previous limits

2010 not permitted in end panels

$a/h > 3$ or $\left[260/\left(h/t_w\right)\right]^2$

$2A_w/\left(A_{fc}+A_{ft}\right) > 2.5$

h/b_{fc} or $h/b_{ft} > 6$

TFA - G2.2(b)(1)

2016 permitted interior panels with

$a/h \le 3$

$2A_w/\left(A_{fc}+A_{ft}\right) \le 2.5$

h/b_{fc} and $h/b_{ft} \le 6$

TFA - G2.2(b)(2)

2016 permitted interior panels with

$a/h \le 3$

$2A_w/\left(A_{fc}+A_{ft}\right) > 2.5$

h/b_{fc} or $h/b_{ft} > 6$

$$V_n = 0.6F_yA_w\left(C_{v2}+\frac{1-C_{v2}}{1.15\sqrt{1+\left(a/h\right)^2}}\right)$$

$$V_n = 0.6F_yA_w\left(C_{v2}+\frac{1-C_{v2}}{1.15\left(a/h+\sqrt{1+\left(a/h\right)^2}\right)}\right)$$

The 2010 C_v

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Chapter G Shear

- G2.3 Transverse Stiffeners
  - w/ and w/o TFA requirements combined in to one section
  - w/o TFA stiffeners must be stiffer than for 2010

Same as 2010 Provisions.

$$I_{st2} = \left[\frac{2.5}{\left(a/h\right)^2}-2\right]b_pt_w^3 \ge 0.5b_pt_w^3$$

$$\left(b/t\right)_{st} \le 0.56\sqrt{\frac{E}{F_{yst}}}$$

$$I_{st1} = \frac{h^4\rho_{st}^{1.3}}{40}\left(\frac{F_{yw}}{E}\right)^{1.5}$$

For full post buckling strength

$$I_{st} \ge I_{st2}+\left(I_{st1}-I_{st2}\right)\rho_w$$

$$\rho_w = \left[\frac{V_r-V_{c2}}{V_{c1}-V_{c2}}\right]$$

Shear strength calculated with 2010 Provisions.

Shear strength calculated with 2016 post-buckling or TFA provisions.

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Chapter H Interaction

- Throughout chapter clarified that required strengths were to be determined in accordance with Chapter C.
  - H1. Doubly and Singly Symmetric Members
    - Deleted limit $0.1 \leq \left(I_{yc}/I_y\right) \leq 0.9$
  - H1.3 Single Axis Flexure
    - Clarified that the flexural strength to be used for in-plane instability was based only on the limit state of yielding.

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Chapter I Composite

- I1.2 Nominal strength determination
  - Added two new approaches
    - (c) Elastic Stress Distribution
    - (d) Effective Stress-Strain Method
- I1.3 Material limitations
  - Increased maximum reinforcing steel to 80 ksi
  - Increased maximum structural steel to 75 ksi

(to be used in calculations)

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Chapter I Composite

- I1.5 Stiffness for Calculation of Required Strengths
  - This section is added to coordinate with Chapter C
  - For flexure; $EI_{eff} = E_s I_s + E_s I_{sr} + C_1 E_c I_c$ (encased)
  - For flexure; $\tau_b = 0.8$
  - For axial; summation of axial stiffnesses

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Chapter I Composite

- I2 Axial Force
  - Effective stiffness of encased composite section increased

- I2 Axial Force
  - Effective stiffness of filled composite section

For 2010 this ratio did not include A_{sr}

$$EI_{eff} = E_s I_s + E_s I_{sr} + C_1 E_c I_c$$

$$C_1 = 0.25 + 3\left(\frac{A_s + A_{sr}}{A_g}\right) \leq 0.7$$

$$EI_{eff} = E_s I_s + E_s I_{sr} + C_3 E_c I_c$$

$$C_3 = 0.45 + 3\left(\frac{A_s + A_{sr}}{A_g}\right) \leq 0.9$$

$0.12 \leq C_1 \leq 0.3$ ← 2010 limits → $0.62 \leq C_3 \leq 0.9$

May now use all of reinforcing steel stiffness, was 0.5

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Chapter I Composite

- I3. Flexure
  - I3.2d Load transfer between steel beam and concrete slab
    - The effect of ductility (slip capacity) of the shear connection at the interface of the concrete slab and steel beam shall be considered.

The intent is to limit the minimum composite action used. Currently the AISC *Manual* tables go from full composite action to 25% composite action. The Commentary provides guidance.

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Chapter J Connections

- J1. General Provisions
  - J1.6. For heavy sections, removed magnetic particle and dye penetrant methods of assessment for access holes
  - J1.8. Clarified/expanded use of bolts in combination with welds
  - J1.10. Deleted. Removed requirement for use of pretensioned bolts in buildings over 125 ft. and moved remaining requirements to J3.1

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Chapter J Connections

- J2. Welds
  - J2.1a. PJP groove welds permitted/requirements when filled less than full depth
  - J2.2b. Fillet weld terminations now presented as a performance requirement
  - J.2.4. Strength. Must account for strain compatibility. The Instantaneous Center of Rotation method is removed and placed in a User Note and the *Manual*

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Chapter J Connections

- J3. Bolts
  - J3.1. High-Strength Bolts
    - Group A – ASTM F3125/F3125M Grades A325, A325M, F1852, and ASTM A354 Grade BC (120 ksi)
    - Group B – ASTM F3125/F3125M Grades A490, A490M, F2280, and ASTM A354 Grade BD (150 ksi)
    - Group C – ASTM F3043 and F3111 (200 ksi)
    - A449 was removed from Group A so strength is now determined as for threaded parts.

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Chapter J Connections

- J3. Bolts
  - J3.1. High-Strength Bolts
    - Designation of when to use which type of installation;
      - J3.1(a) snug-tight
        Two cases
      - J3.1(b) pre-tensioned
        Three cases
      - J3.1(c)slip critical
        Two cases

This change is more for clarity than for actually changing anything.

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Chapter J Connections

- J3.2 Holes
  - For bolts 1 in. diameter and larger the standard hole size is now 1/8 in. larger than the bolt to address fit-up issues.
  - Specific requirements for washers removed and reference made to RCSC requirements.
- J3.3 Minimum Spacing
  - Clear distance not less than *d*

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Chapter J Connections

- J3.10 Bearing and Tearout Strength
  - The format of these equations has been changed to present bearing and tearout in separate subsections.
  - The strength equations are unchanged
  - The process
    - Determine bearing strength of each bolt
    - Determine tearout strength of each bolt

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Chapter J Connections

- J4.1 Affected elements in tension
  - Special requirement for bolted splice plates removed since the resistance/safety factor (unchanged) already accounts for this

$$\cancel{A_e = A_n \leq 0.85A_g}$$

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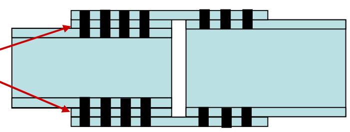
Chapter J Connections

- J5.2 Fillers
  - This section now defined as specifically for bearing type connections
  - Fillers may now be welded
  - Fillers in slip critical connections and turn of the nut method removed as a special requirement.



Chapter J Connections

- J10.3 Web crippling
  - The variable Q_f is introduced to permit material from Chapter K for HSS to be moved here.

$$R_n = 0.80 t_w^2 \left[1 + 3 \left(\frac{l_b}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{E F_{yw} t_f}{t_w}} Q_f \qquad \text{(J10-4)}$$

- J10.5 Web compression buckling
  - Added Q_f

$$R_n = \frac{24 t_w^3 \sqrt{E F_{yw}}}{h} Q_f \qquad \text{(J10-8)}$$

Chapter J Connections

- J10.6 Web panel zone shear
  - Variable P_r changed to αP_r for consistency with other parts of the Specification considering LRFD and ASD.
  - Clarified that consideration of panel zone deformation means inelastic panel zone deformation
- J10.10 Transverse Forces on Plates
  - Section added with general provisions and a user note

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Chapter K HSS Connections

- Design of HSS Connections
  - Clarifies that box sections must be of uniform thickness to use this chapter
  - Clarifies that you can design connections that do not meet the limitations, just can not use these specific provisions
  - As many provisions as appropriate now simply refer to Chapter J

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Remaining Chapters

- Chapter L: Serviceability
  - Limit states now specifically referenced
  - Camber deleted since it is not a serviceability limit state
- Chapter M: Fabrication and Erection
  - Only editorial and clarifications

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Remaining Chapters

- Chapter N: Quality
  - Steel Deck requirements now point to SDI
  - Welds
    - Access holes in heavy shapes
    - Unauthorized welds
  - N5.5 added establishment of ultrasonic testing rejection rate
  - N5.7 added for galvanized main members
  - N6 Composite deleted since reference SDI

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Appendix 1

- Title changed from *Design by Inelastic Analysis* to *Design by Advanced Analysis*
- Added a new section for design by elastic analysis
  - Includes direct modeling of system and member imperfections.
  - Essentially results in designing with L_c=0.

$$P_n = F_y A_g \quad \text{or} \quad P_n = F_y A_e$$

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Appendix 2

- Design for Ponding
  - Clearly identify the assumptions which must be met for use of the appendix method.
    - Flat roofs with rectangular bays where beams are uniformly spaced and girders are considered to be uniformly loaded.
  - Clarified load determination for use of Appendix 2.2 Improved Design for Ponding

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Appendix 3

- Fatigue
  - Simplifies some equations, adds new fatigue cases and revises figures
  - Changed the requirement to check fatigue to a positive; check when cycles exceed 20,000
  - Many changes throughout

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Appendix 4

- Fire
  - Clarify that design by analysis is acceptable under the alternative methods provisions of the applicable building code
  - Added information on bolt strength
  - Added reference to newly published standards, NFPA 557 and SFPE S.01
  - Added composite beam strength retention factors
  - Added provisions for shear and combined forces and torsion

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Appendix 5

- Existing Structures
  - Removed limit to gravity loading only
  - Gravity loading limit retained for load testing
  - Added the requirement to record deformations once the loading has been removed

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Appendix 6

- Stability Bracing
  - Name Changes:
    - Nodal bracing changed to Point bracing
    - Relative bracing changed to Panel bracing
  - Clarify the "In lieu of this appendix" requirements
  - Slight revision in required strength values
  - Beam-Column bracing provisions expanded for doubly and singly symmetric members

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Appendix 7

- Alternative Methods for Stability
  - 7.3 First-Order analysis
    - Change the limits of applicability to account for inelasticity in a member with slender elements,

      $\alpha P_r \le 0.5 P_y$ becomes $\alpha P_r \le 0.5 P_{ns}$
    - Now permits the use of effective length less than actual length between brace points (K<1)

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Appendix 8

- Approximate Second-Order Analysis
  - Deleted the phrase "*as an alternative to a rigorous second-order analysis*" from the preamble.
  - Recognized as an approximate procedure.
    - Has no impact on use of the method since this is simply a method, not rigorous, not non-rigorous.
    - Appendix 8 is referenced in Chapter C as an acceptable method.

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Conclusion

- AISC Board approved June 16, 2016
- ANSI approved July 7, 2016
- AISC 360-16 issued in print in January 2017.
- 15th Edition *Steel Construction Manual* available Summer 2017.

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Thank You

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