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Course Description

June 12, 2017 – Fundamentals of Stability for Steel Design: Design of Compression Members

Initially, an overview of flexural, torsional, and flexural-torsional resistance of individual column members will be provided. Emphasis then will be placed on defining and assessing the AISC LRFD and ASD strengths of various structural shapes, including wide flange, round and square HSS, cruciform, equal and unequal single and double leg angles, WT, channel, and built-up shapes.

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Learning Objectives

- Explain the design limit states of flexural, torsional, and flexural-torsional buckling resistance of compression members
- Identify the steps to determine the design strengths for column design using the AISC *Specification*.
- Identify the applicable buckling limit states for various structural shapes subjected to compression.
- Understand stability design through the use of design examples.

There's always a solution in steel.

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Fundamentals of Stability for Steel Design

Session 2: Design of Compression Members

June 12, 2017

Presented by
Perry S. Green, P.E., Ph.D.
Bechtel Corporation, Waynesboro, GA

There's always a solution in steel.

Fundamentals of Stability for Steel Design

Session 2
Design of Compression Members

Perry S. Green, P.E., Ph.D.

structural STEEL

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SESSION OUTLINE

- Introduction
- Flexural Buckling
- Torsional Buckling
- Flexural-Torsional Buckling
- Built-Up Members
- Design Examples:
  - Flexural Bucking
  - Torsional Buckling
  - Flexural-Torsional Buckling

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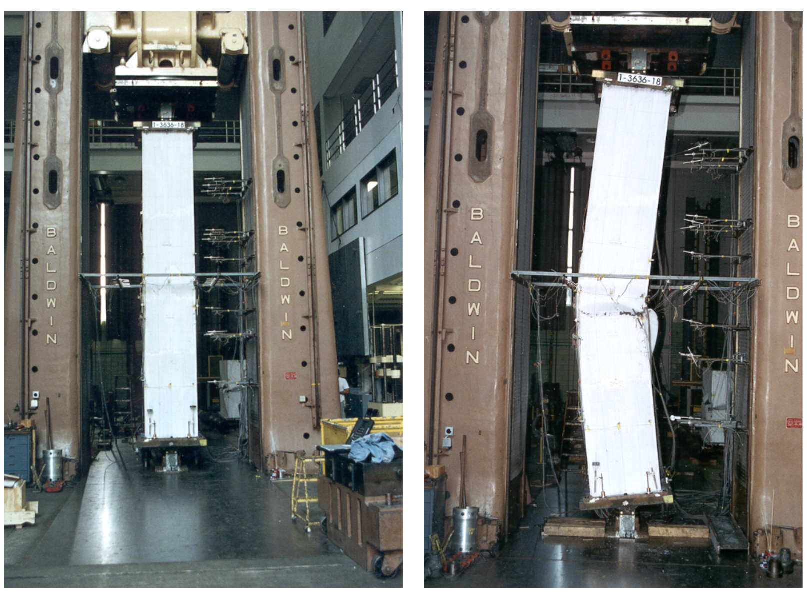
INTRODUCTION

COMPRESSION MEMBERS
FLEXURAL BUCKLING



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2010 AISC SPECIFICATION
CHAPTER E - COMPRESSION MEMBERS

- **All compression member strengths are controlled by the limit state of *buckling***

Bad News

There are four different types of buckling

FB: Flexural Buckling
TB: Torsional Buckling
FTB: Flexural-Torsional Buckling
LB: Local Plate Buckling

Good News

Not all types are applicable to every member

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ANSI/AISC 360-10 *Specification for the Design of Steel Buildings*, 2010

CHAPTER E
DESIGN OF MEMBERS FOR COMPRESSION

E1. GENERAL PROVISIONS
E2. EFFECTIVE LENGTH
E3. FLEXURAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS
E4. TORSIONAL AND FLEXURAL-TORSIONAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS
E5. SINGLE ANGLE COMPRESSION MEMBERS
E6. BUILT-UP MEMBERS
E7. MEMBERS WITH SLENDER ELEMENTS

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2010 AISC SPECIFICATION
CHAPTER E - COMPRESSION MEMBERS

FB: Flexural Buckling

TB: Torsional Buckling

FTB: Flexural-Torsional Buckling

~~**LB: Local Plate Buckling**~~

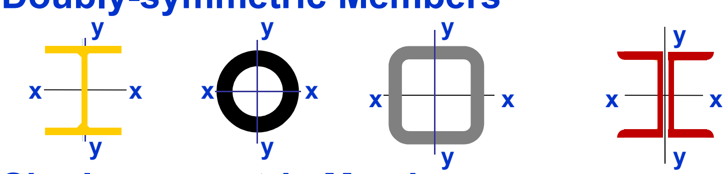
only for members with slender plate elements

TABLE USER NOTE E1.1
Selection Table for the Application of Chapter E Sections

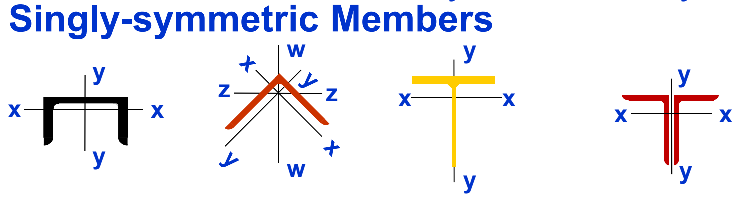
Cross Section	Without Slender Elements: Sections in Chapter E	Without Slender Elements: Limit States
	E3 E4	FB TB
	E3 E4	FB FTB
	E3	FB
	E3	FB
	E3 E4	FB FTB
	E3 E4 E6	FB FTB
	E5	FB TB
	E3	FB
Unsymmetrical shapes other than single angles	E4	FTB

TYPES OF COMPRESSION MEMBERS

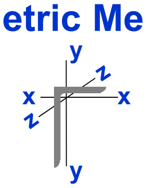
- **Doubly-symmetric Members**



- **Singly-symmetric Members**



- **Unsymmetric Members**



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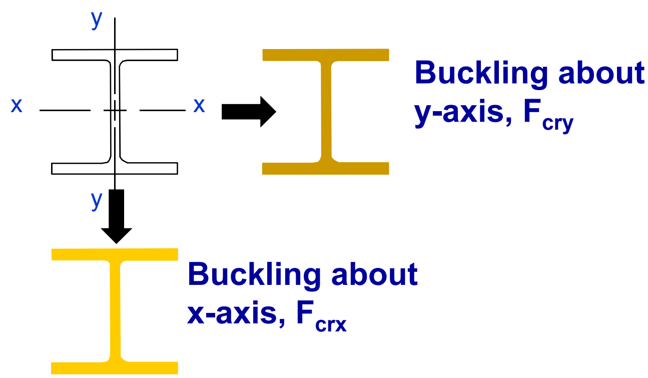
FLEXURAL BUCKLING
DOUBLY-SYMMETRIC MEMBERS



Most cases in design of W Shapes involve flexural buckling

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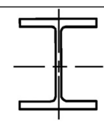
2010 AISC SPECIFICATION
CHAPTER E - COMPRESSION MEMBERS

CROSS SECTION	WITHOUT SLENDER ELEMENTS	
	SECTIONS IN CHAPTER E	LIMIT STATES
	E3 E4	FB TB

FB: Flexural Buckling

TB: Torsional Buckling

FTB: Flexural-Torsional Buckling

For doubly-symmetric I shapes *flexural buckling* will always control if twist of the member is prevented at all supports and brace points, *i.e.,* the lateral unbraced length ≥ torsional unbraced length

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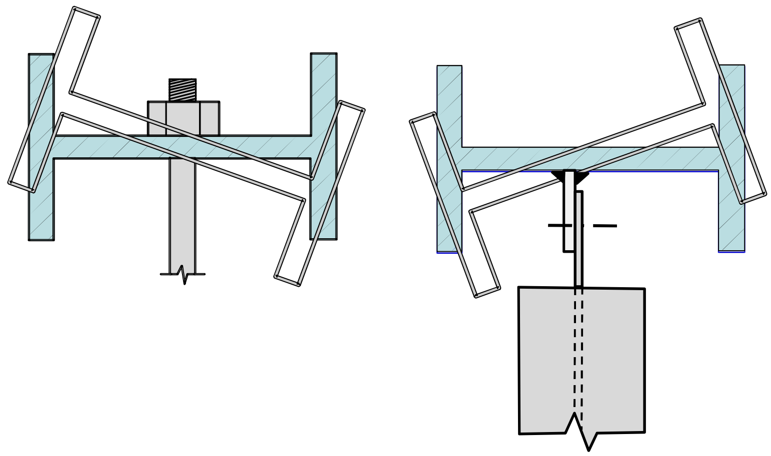
INEFFECTIVE TORSIONAL SUPPORT



Torsional Buckling will be discussed later

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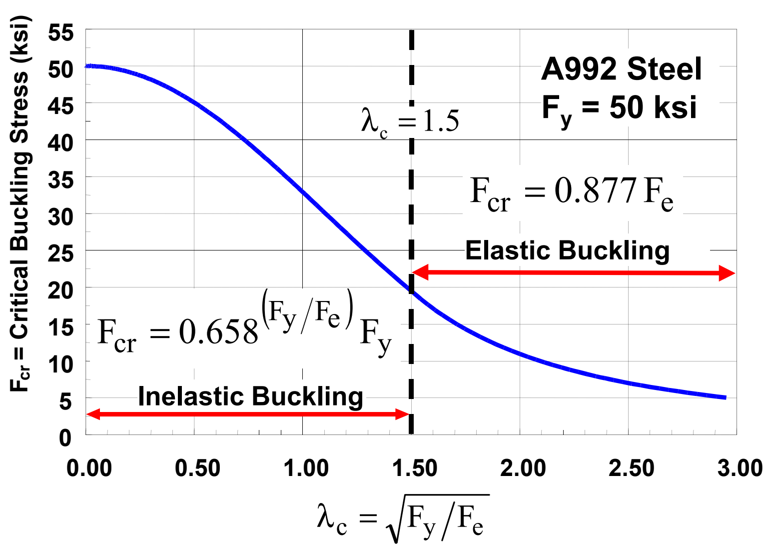
COMPRESSION MEMBERS

FLEXURAL BUCKLING W/O SLENDER ELEMENTS



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COMPRESSION MEMBERS

FLEXURAL BUCKLING W/O SLENDER ELEMENTS

ANSI/AISC 360-10 *Specification for the Design of Steel Buildings*, 2010

$$P_n = F_{cr} A_g \qquad (E3-1)$$

$$\frac{KL}{r} \le 4.71\sqrt{\frac{E}{F_y}}$$

or

$$\frac{F_y}{F_e} \le 2.25$$

YES

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y \qquad (E3-2)$$

NO

$$F_{cr} = 0.877 F_e \qquad (E3-3)$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \qquad (E3-4)$$

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COMPRESSION MEMBER DESIGN EXAMPLE 1

ANSI/AISC 360-10 *Specification for the Design of Steel Buildings*, 2010

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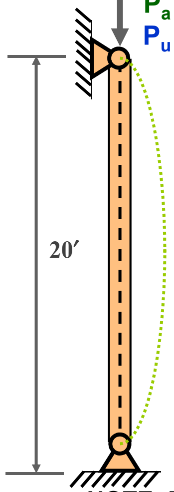
WF MEMBER DESIGN STRENGTH

P_D = 125 kips
P_L = 400 kips
P_a = 525 kips
P_u = 790 kips



Check Required Strength W12 x 96

A_g = 28.2 in.^{2} I_y = 270 in.^{4} r_y = 3.09 in.

F_y = 50 ksi KL_y = 240 in.

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 (29000)}{\left(\frac{240}{3.09}\right)^2} = 47.44\,\text{ksi} \qquad (E3-4)$$

$$\frac{F_y}{F_e} = \frac{50}{47.44} = 1.05 \le 2.25 \qquad \therefore \text{Inelastic}$$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y = \left[0.658^{1.05}\right](50) = 32.17\,\text{ksi} \quad (E3-2)$$

$$P_n = F_{cr} A_g = (32.17)(28.2) = 907.0\ \text{kips} \qquad (E3-1)$$

$$\frac{P_n}{\Omega_c} = \frac{907}{1.67} = 543\ \text{kips} > 525\ \text{kips} \qquad \therefore \text{OK}$$

$$\phi_c P_n = 0.9(907) = 816\ \text{kips} > 790\ \text{kips} \qquad \therefore \text{OK}$$

NOTE: From the Column Strength Tables $\phi_c P_n$ = 816 kips

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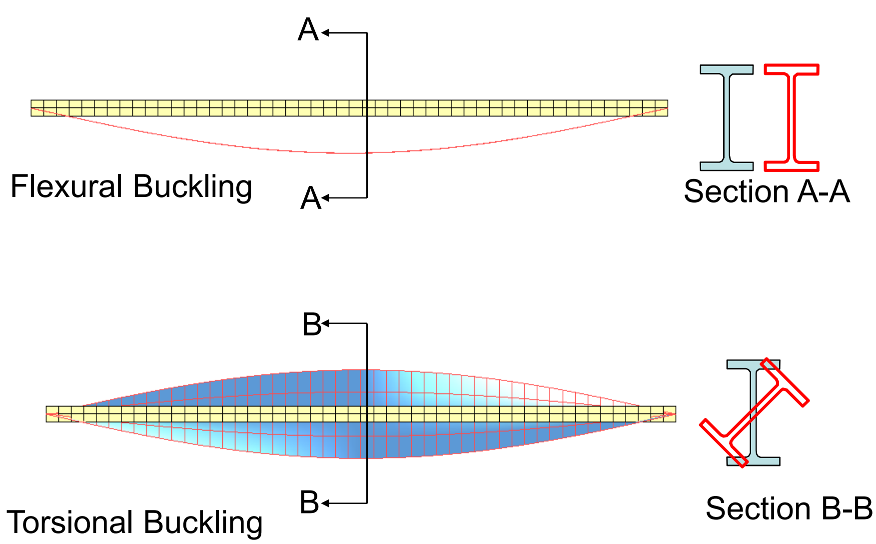
POTENTIAL BUCKLING MODES



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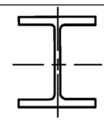
2010 AISC SPECIFICATION
CHAPTER E - COMPRESSION MEMBERS

CROSS SECTION	WITHOUT SLENDER ELEMENTS	
	SECTIONS IN CHAPTER E	LIMIT STATES
	E3 E4	FB TB

FB: Flexural Buckling

TB: Torsional Buckling

FTB: Flexural-Torsional Buckling

Torsional buckling occurs over short lengths

- Usually KL/r will be less than approximately 50
- Doubly-symmetric section
- Wide flange, cruciform, and point symmetric sections

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TORSIONAL BUCKLING
OPEN CROSS SECTION MEMBERS

- **Buckling in a torsional mode (torsion of non-circular sections involves torsional shear and warping:**
  - K_z is normally taken as 1.0
  - Effective length in torsion is $K_zL_z = L_z$
  - Function of the torsional constant $J \approx \Sigma(bt^3/3)$
  - Capacity of section with warping stiffness is a function of warping constant C_w
  - C_w, J, r_x, r_y are given in the Properties Tables where x and y are the axes of symmetry of the section
  - E = 29,000 ksi, G = 0.385E (11,200 ksi)

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COMPRESSION MEMBERS

TORSIONAL BUCKLING W/O SLENDER ELEMENTS

ANSI/AISC 360-10 *Specification for the Design of Steel Buildings*, 2010

$$P_n = F_{cr} A_g \qquad (E4-1)$$

$$\frac{F_y}{F_e} \le 2.25$$

YES

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y \qquad (E3-2)$$

$$F_e = \left[\frac{\pi^2 E C_w}{(K_z L)^2} + GJ\right] \frac{1}{I_x + I_y} \qquad (E4-4)$$

NO

$$F_{cr} = 0.877 F_e \qquad (E3-3)$$

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COMPRESSION MEMBER DESIGN EXAMPLE 2

ANSI/AISC 360-10 *Specification for the Design of Steel Buildings*, 2010

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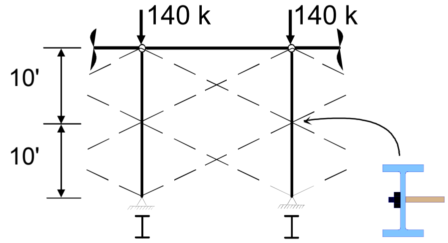
TORSIONAL BUCKLING EXAMPLE



Are the columns stable with the 140 k factored load if bracing at midheight is supplied by tie rods?

Columns: W16x26; F_y = 50 ksi; P_y=384 kips

I_y = 9.59 in^4; J = 0.26 in^4; A_g = 7.68 in^2

I_x = 301 in^4; C_w = 565 in^6

Flexural Buckling Effective Length: L_y = 10 ft

ϕP_n =150 kips > 140 kips $\therefore$OK

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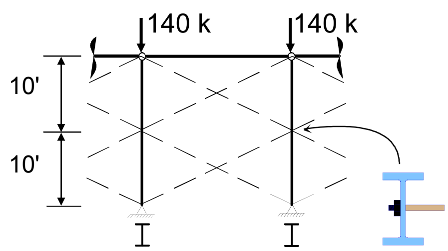
TORSIONAL BUCKLING EXAMPLE



Torsional Buckling

Effective Length: L_T = 20 ft.

$$F_e = \left[\frac{\pi^2\ (29000)(565)}{(20\ \text{x}\ 12)^2} + 11200\,(0.262)\right]\frac{1}{(301+9.59)} = 18.5\ \text{ksi} \quad (\text{E4}-4)$$

$$\frac{F_y}{F_e} = \frac{50}{18.5} = 2.70 > 2.25 \quad \therefore \text{Elastic} \quad F_{cr} = 0.877\,(18.5) = 16.2\ \text{ksi} \quad (\text{E3}-3)$$

$$P_{Tn} = F_{cr} A_g = (16.2)(7.68) = 124.4\ \text{kips} \quad (\text{E4}-1)$$

$$\phi_c P_{Tn} = 0.9\,(124.4) = 112\ \text{kips} < 140\ \text{kips} \quad \therefore \text{NG}$$

Note: If L_T = 10 ft., P_T = 283 k

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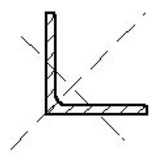
2010 AISC SPECIFICATION
CHAPTER E - COMPRESSION MEMBERS

CROSS SECTION	WITHOUT SLENDER ELEMENTS	
	SECTIONS IN CHAPTER E	LIMIT STATES
	E5	FB TB

FB: Flexural Buckling
TB: Torsional Buckling
FTB: Flexural-Torsional Buckling

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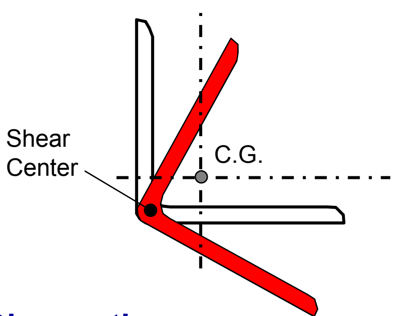
TORSIONAL BUCKLING
EQUAL LEG ANGLE MEMBERS

$$P_{crz} = \frac{GJA}{I_P}$$

$$A = 2bt, \quad J = 2\frac{bt^3}{3}$$

$$I_P = 2\frac{b^3 t}{3}, \quad C_w \approx 0$$



$$P_{crz} = \frac{2Gt^3}{b}$$

Observations:

Torsional buckling is not a function of the angle length

The buckling capacity decreases for large leg sizes

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TORSIONAL BUCKLING
EQUAL LEG ANGLE MEMBERS

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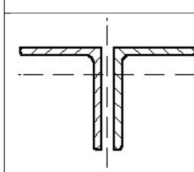
2010 AISC SPECIFICATION
CHAPTER E - COMPRESSION MEMBERS

CROSS SECTION	WITHOUT SLENDER ELEMENTS	
	SECTIONS IN CHAPTER E	LIMIT STATES
	E3 E4 E6	FB FTB

FB: Flexural Buckling

TB: Torsional Buckling

FTB: Flexural-Torsional Buckling

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FLEXURAL-TORSIONAL BUCKLING
SINGLY AND UNSYMMETRIC MEMBERS

- Flexural buckling will occur in the plane of symmetry
- Sections always rotate about the shear center; the shear center lies on the axis of symmetry
- Singly symmetric sections can buckle either in a Flexural or Flexural-Torsional mode
- Flexural-Torsional buckling will occur by twisting about a point in the plane of symmetry
- Unsymmetric sections will always buckle in a Flexural-Torsional mode

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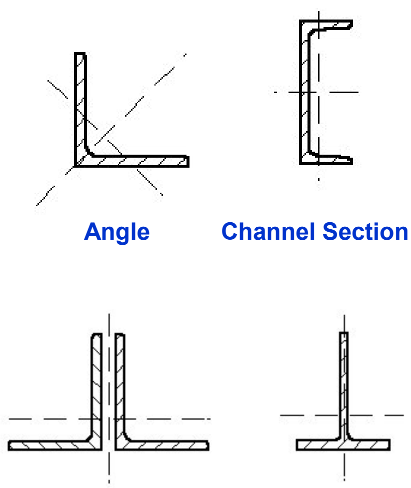
FLEXURAL-TORSIONAL BUCKLING
SINGLY AND UNSYMMETRIC MEMBERS



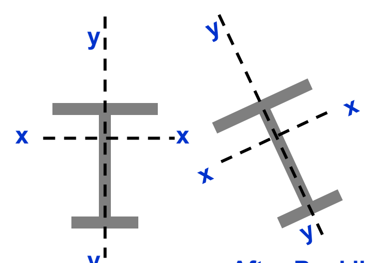
Angle

Channel Section

Double Angle
Tee Section

WT Section

Twist and lateral
movement
after buckling



Before Buckling

After Buckling

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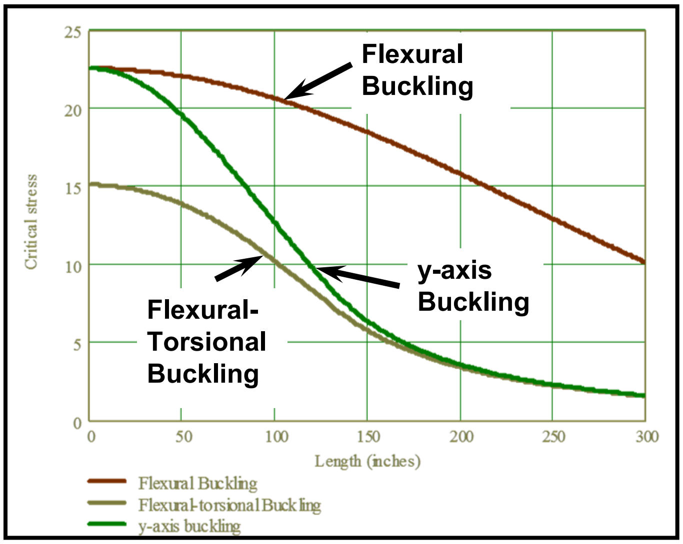
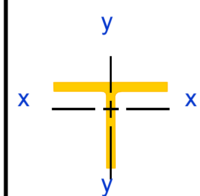
FLEXURAL-TORSIONAL BUCKLING
SINGLY AND UNSYMMETRIC MEMBERS

INELASTIC BUCKLING WT 6 X 7





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POLLING QUESTION 1

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COMPRESSION MEMBERS

BUCKLING OF SINGLE ANGLES WITH OR W/O SLENDER ELEMENTS

AISC/LRFD *Specification for Single Angle Members* (Nov. 10, 2000) permitted design for L/ r_z only. OK for usual dimensions, but can be unconservative for short single angles

The 2005 AISC Specification recommended using effective length formulas from ASCE Transmission Line Standard for single angle columns attached to a gusset plate to one leg

See the SSRC Guide to Stability Design Criteria for Metal Structures Chapter 11 for more info

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COMPRESSION MEMBERS

BUCKLING OF SINGLE ANGLES WITH OR W/O SLENDER ELEMENTS

- **The effects of eccentricity on single angle members are permitted to be neglected when the members are evaluated as axially loaded compression members using one of the effective slenderness ratios specified below, provided that:**
  - Members are loaded at the ends in compression through the same one leg
  - Members are attached by welding or by minimum two-bolt connections; and
  - No intermediate transverse loads are present

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COMPRESSION MEMBERS
EQUIVALENT KL/r FOR SINGLE ANGLES

Case 1 - E5(a): Planar truss, attachment to chord or gusset plate

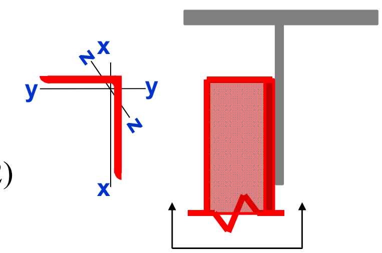
$$\frac{L}{r_x} \le 80$$

YES:

$$\frac{KL}{r} = 72 + 0.75\frac{L}{r_x} \qquad \text{(E5-1)}$$

NO:

$$\frac{KL}{r} = 32 + 1.25\frac{L}{r_x} \qquad \text{(E5-2)}$$



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COMPRESSION MEMBERS
EQUIVALENT KL/r FOR SINGLE ANGLES

Additional requirements for unequal leg angles:

$$\left(\frac{KL}{r}\right)_{eq} \ge 0.95\frac{L}{r_z}$$

For unequal leg angles with b_{long}/b_{short} < 1.7 where attachment is to the shorter leg, increase

$$\left(\frac{KL}{r}\right)_{eq} \text{ by } 4\left[\left(\frac{b_{long}}{b_{short}}\right)^2 - 1\right]$$

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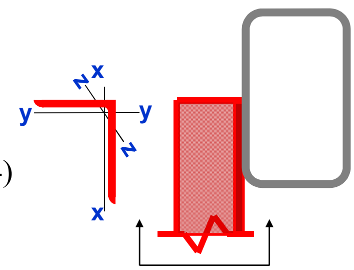
COMPRESSION MEMBERS
EQUIVALENT KL/r FOR SINGLE ANGLES

Case 2 - E5(b): Box or space truss

$\frac{L}{r_x} \le 75$

YES: $\frac{KL}{r} = 60 + 0.80\frac{L}{r_x}$ (E5−3)

NO: $\frac{KL}{r} = 45 + \frac{L}{r_x}$ (E5−4)



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COMPRESSION MEMBERS
EQUIVALENT KL/r FOR SINGLE ANGLES

Additional requirements for unequal leg angles:

$$\left(\frac{KL}{r}\right)_{eq} \ge 0.82\frac{L}{r_z}$$

For unequal leg angles with b_{long}/b_{short} < 1.7 where attachment is to the shorter leg, increase

$$\left(\frac{KL}{r}\right)_{eq} \text{ by } 6\left[\left(\frac{b_{long}}{b_{short}}\right)^2 - 1\right]$$

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COMPRESSION MEMBER DESIGN EXAMPLE 3

ANSI/AISC 360-10 *Specification for the Design of Steel Buildings*, 2010

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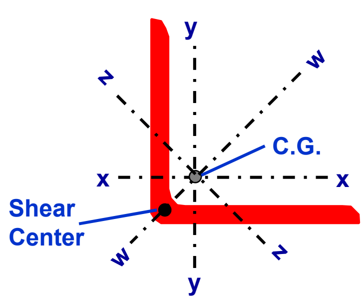
MEMBER DESIGN STRENGTH
SINGLE ANGLE COLUMN ∟2 x 2 x 3/16



An equal leg angle is welded to the web of tee chords of a planar truss. Is it more advantageous to use 50 ksi material rather than 36 ksi yield strength material?

Section Properties:

$A = 0.722$ in.^{2}
$r_x = 0.612$ in.
$r_z = 0.389$ in.
$Q_s = 1.0$

Compression Member Length: L = 44 in.

Material Properties:

$F_y = 36$ ksi or 50 ksi
E = 29,000 ksi
G = 0.385 E

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MEMBER DESIGN STRENGTH
SINGLE ANGLE COLUMN ∟2 x 2 x 3/16

Effective Slenderness

$$\frac{b}{t}=\frac{2}{0.1875}=10.7<20$$

$$\frac{L}{r_x}=\frac{44}{0.612}=72<80$$

Details:
Equal leg angle
One leg connected to tee chord
Attached by welding
Width-to-thickness ratio meets criteria in Section E5
F_y = 36 ksi

$$\left(\frac{KL}{r}\right)_x=72+0.75\frac{L}{r_x}=126\ \leq 4.71\sqrt{\frac{E}{F_y}}=133 \qquad (E5-1)$$

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MEMBER DESIGN STRENGTH
SINGLE ANGLE COLUMN ∟2 x 2 x 3/16

$$F_e=\frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}=18.03\,\text{ksi} \qquad (E3-4)$$

$$F_{cr}=\left[0.658^{\frac{F_y}{F_e}}\right]F_y=15.60\,\text{ksi} \qquad (E3-2)$$

$$P_n=F_{cr}A_g=11.27\,\text{kips} \qquad (E3-1)$$

➡ ASD $\frac{P_n}{\Omega}=6.75\,\text{kips}$ LRFD $\phi P_n=10.14\,\text{kips}$

Single angle will buckle inelastically in a flexural mode

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MEMBER DESIGN STRENGTH

SINGLE ANGLE COLUMN ∟2 x 2 x 3/16

Effective Slenderness

$$\frac{b}{t} = \frac{2}{0.1875} = 10.7 < 20$$

Details:
All same except
F_y = 50 ksi

$$\frac{L}{r_x} = \frac{44}{0.612} = 72 < 80$$

$$\left(\frac{KL}{r}\right)_x = 72 + 0.75\frac{L}{r_x} = 126 \; > 4.71\sqrt{\frac{E}{F_y}} = 113 \qquad (E5-1)$$

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MEMBER DESIGN STRENGTH

SINGLE ANGLE COLUMN ∟2 x 2 x 3/16

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = 18.03\,\text{ksi} \qquad (E3-4)$$

$$F_{cr} = 0.877\,F_e = 15.81\,\text{ksi} \qquad (E3-2)$$

$$P_n = F_{cr} A_g = 11.41\,\text{kips} \qquad (E3-1)$$

➡ ASD $\frac{P_n}{\Omega} = 6.83\,\text{kips}$ LRFD $\phi P_n = 10.27\,\text{kips}$

Single angle will buckle elastically in a flexural mode
NOTE: Design strength has only increased about 1%

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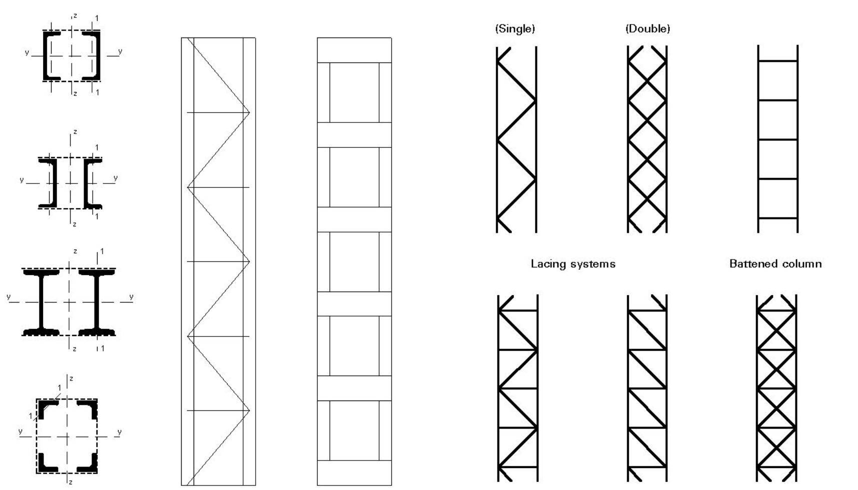
COMPRESSION MEMBERS
BUILT-UP COLUMNS

- **Built-up members are comprised of two or more sections**
  - Stitch bolts
  - Batten plates
  - Lacing
  - Combined batten plates and lacing
  - Perforated cover plates
- **Built-up batten member buckling is somewhat similar to frame buckling**
  - Battens act like beams
  - Battens get shear and moment due to the bending of the frame like built-up member at the time of buckling

COMPRESSION MEMBERS
BUILT-UP LACED OR BATTEN COLUMNS



See the SSRC Guide to Stability Design Criteria for Metal Structures Chapter 3 for more info

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COMPRESSION MEMBERS

BUCKLING OF BUILT-UP MEMBERS COMPOSED OF TWO SHAPES

(a) For intermediate connectors that are bolted snug-tight

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \qquad (E6-1)$$

a = distance between connectors (in.)

r_i = minimum radius of gyration of individual component (in.)

$\left(\frac{KL}{r}\right)_m$ = modified slenderness of built-up member

$\left(\frac{KL}{r}\right)_o$ = slenderness ratio of built-up member acting as a unit in the buckling direction being considered

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COMPRESSION MEMBERS

BUCKLING OF BUILT-UP MEMBERS COMPOSED OF TWO SHAPES

(b) For intermediate connectors that are welded or are connected by means of pretensioned bolts

$\frac{a}{r_i} \le 40$?

YES:

$$\left(\frac{KL}{r}\right)_m = \left(\frac{KL}{r}\right)_o \qquad (E6-2a)$$

NO:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{K_i a}{r_i}\right)^2} \qquad (E6-2b)$$

K_i = 0.50
= 0.75
= 0.86 all others

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COMPRESSION MEMBER DESIGN EXAMPLE 4

ANSI/AISC 360-10 *Specification for the Design of Steel Buildings*, 2010

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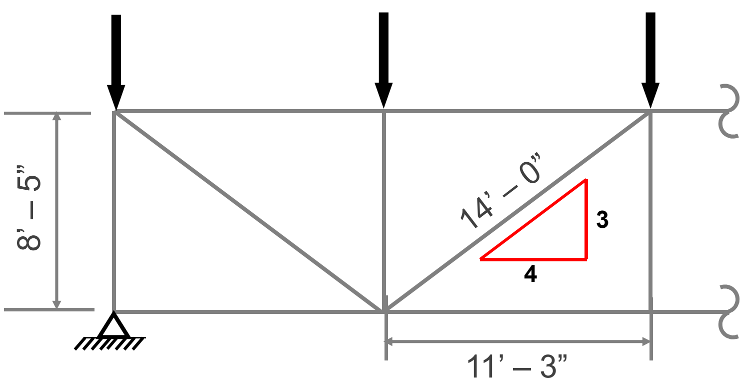
MEMBER DESIGN STRENGTH
DIAGONAL BRACE 2∟s 6 x 4 x 5/8 LLBB

What is the design strength of the 14 ft. long diagonal brace that is comprised of 2L's 6 x 4 x 5/8 LLBB, A36 steel.

There are two intermediate bolted connectors at a spacing of 56 in. Use K = 1.0



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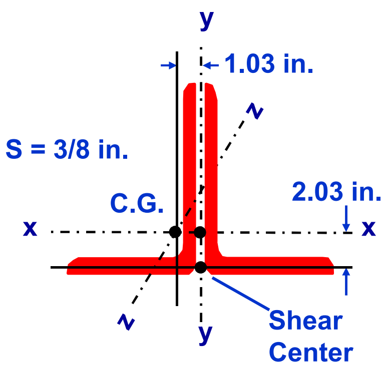
MEMBER DESIGN STRENGTH

DIAGONAL BRACE 2∟s 6 x 4 x 5/8 LLBB



Single Angle Section Properties

$J = 0.775$ in.^{4} $C_w = 1.59$ in.^{6}

$r_z = 0.859$ in. $r_{yi} = 1.13$ in.

Double Angle Section Properties

$A = 11.7$ in.^{2} $I_x = 42.0$ in.^{4}

$r_x = 1.89$ in. $r_y = 1.66$ in.

$\bar{r}_0 = 3.05$ in. $J = 1.55$ in.^{4}

$H = 0.684$ in.

Material Properties

$F_y = 36$ ksi
$E = 29{,}000$ ksi
$G = 0.385\,E$

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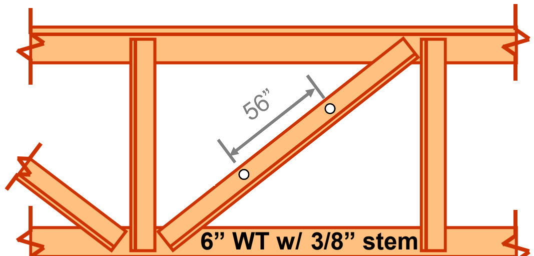
MEMBER SLENDERNESS

DIAGONAL BRACE 2∟s 6 x 4 x 5/8 LLBB



x – x axis: $KL/r_x = 88.9$

z – z axis: $a/r_z = 65.2$

y – y axis: $KL/r_y = 101.2$

The intermediate connectors are only snug-tight
Determine the design strength neglecting flexural-torsional buckling;
Y-Y axis is built-up and acts as a single unit, therefore,

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \qquad \text{(E6-1)}$$

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MEMBER DESIGN STRENGTH
DIAGONAL BRACE 2∟s 6 x 4 x 5/8 LLBB

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} = \sqrt{(101.2)^2 + (65.2)^2} = 120.38$$

$$F_{ey} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)_m^2} = 19.75\,\text{ksi} \qquad F_y/F_{ey} = 36/19.75 = 1.82 < 2.25$$

$$F_{cry} = 0.658^{\frac{F_y}{F_e}} F_y = 16.79\,\text{ksi}$$

$$P_n = A_g F_{cr} = (11.7)(16.79) = 196.4\,\text{kips}$$

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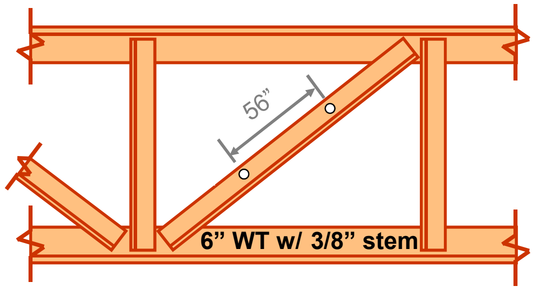
MEMBER SLENDERNESS
DIAGONAL BRACE 2∟s 6 x 4 x 5/8 LLBB



x – x axis: KL/r_x = 88.9
z – z axis: a/r_z = 65.2
y – y axis: KL/r_y = 101.2

$$\frac{a}{r_i} = \frac{56}{0.859} = 65.2 > 40$$

The intermediate connectors are <u>pretensioned</u>
Determine the design strength neglecting flexural-torsional buckling;
Y-Y axis is built-up and acts as a single unit, therefore,

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{K_i a}{r_i}\right)^2} \qquad \text{(E6-2b)}$$

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MEMBER DESIGN STRENGTH

DIAGONAL BRACE 2∟s 6 x 4 x 5/8 LLBB

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{K_i a}{r_i}\right)^2}$$

$$= \sqrt{(101.2)^2 + [(0.5)(65.2)]^2} = 106.3$$

$$F_{ey} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)_y^2} = 25.32\,\text{ksi} \qquad F_y / F_{ey} = 36/25.32 = 1.42 < 2.25$$

$$F_{cry} = 0.658^{\frac{F_y}{F_e}} F_y = 19.85\,\text{ksi}$$

$$P_n = A_g F_{cr} = (11.7)(19.85) = 232.3\,\text{kips}$$ ⬅ ∴Use

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DETERMINATION OF CRITICAL STRESS

DIAGONAL BRACE 2∟s 6 x 4 x 5/8 LLBB

Flexural-Torsional Buckling

$$F_{crz} = \left(\frac{\pi^2 E C_w}{(K_z L)^2} + GJ\right)\frac{1}{A_g \bar{r}_o^2} \cong \frac{GJ}{A_g \bar{r}_o^2} \qquad (E4-3)$$

Small compared to GJ, therefore this term can be safely ignored for angles, double angles and tees. In this case, 32 versus 17360

$$F_{crz} = \frac{GJ}{A_g \bar{r}_o^2} = \frac{(11200)(1.55)}{(11.7)(3.05)^2} = 159.5\,\text{ksi}$$

$$F_{cr} = \frac{F_{cry} + F_{crz}}{2H}\left[1 - \sqrt{1 - \frac{4 F_{cry} F_{crz} H}{(F_{cry} + F_{crz})^2}}\right] \qquad (E4-2)$$

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MEMBER DESIGN STRENGTH

DIAGONAL BRACE 2∟s 6 x 4 x 5/8 LLBB

$$F_{cr} = \frac{19.85 + 159.5}{2(0.684)}\left[1 - \sqrt{1 - \frac{4(19.85)(159.5)(0.684)}{(19.85 + 159.5)^2}}\right]$$

$$F_{cr} = 19.0\,\text{ksi}$$

$$P_n = A_g F_{cr} = (11.7)(19.0) = 222.4\,\text{kips}$$

$$\text{ASD} \quad \frac{P_n}{\Omega} = 133.2\,\text{kips} \qquad \text{LRFD} \quad \phi P_n = 200.10\,\text{kips}$$

The Flexural-Torsional buckling load is ~5% less than the Euler buckling load in this case. If the legs of the angles or stem of the tees are thin, Flexural-Torsional buckling could be up to 20% less than the Flexural buckling load.

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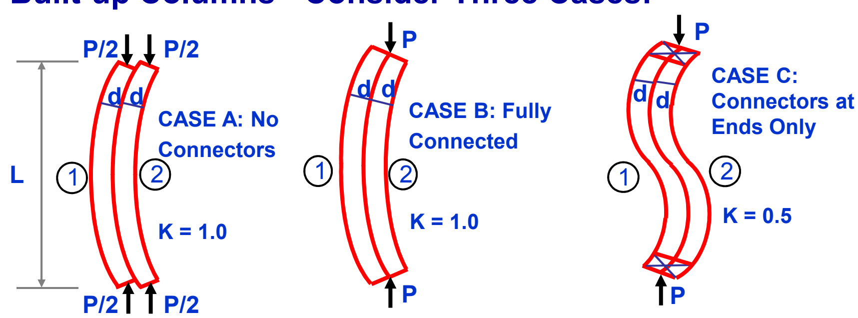
COMPRESSION MEMBERS

BUILT-UP COLUMNS

Effect of Connector End Restraint on the Behavior of Built-up Columns - Consider Three Cases:



$$\frac{P_{cr}}{2} = \frac{\pi^2 E\left(\frac{1}{12} b d^3\right)}{L^2} \qquad P_{cr} = \frac{\pi^2 E\left(\frac{1}{12} b (2d)^3\right)}{L^2} \qquad \frac{P_{cr}}{2} = \frac{\pi^2 E\left(\frac{1}{12} b d^3\right)}{(0.5L)^2}$$

$$P_{cr} = \frac{\pi^2 E b d^3}{6L^2} \qquad P_{cr} = \frac{4}{6}\frac{\pi^2 E b d^3}{L^2} \qquad P_{cr} = \frac{4}{6}\frac{\pi^2 E b d^3}{L^2}$$

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COMPRESSION MEMBERS
BUILT-UP COLUMNS

CASE A: With no connectors, I_{eff} of the cross-section is the sum of the I of the individual components,
$I_{eff} = 2\ [^1/_{12}\ bd^3]$.

CASE B: With the two components fully connected, no slip occurs between members, the cross-section acts as a single unit and $I_{eff} = {}^1/_{12}\ b(2d)^3$ or four times that with no connectors.

CASE C: Providing just a few connectors will give an effective member stiffness somewhere between these two extremes. The maximum effect will occur when connectors are placed near the ends where the relative displacement between the contact surfaces would be a maximum. Connections near midspan will have no effect.

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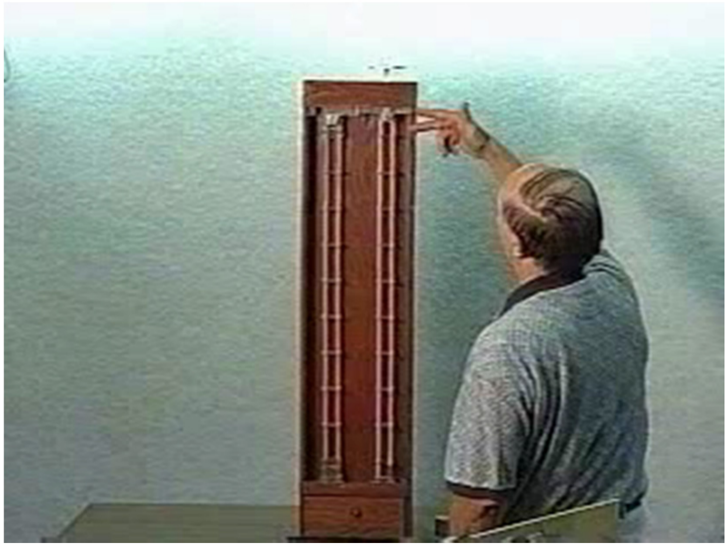
COMPRESSION MEMBERS
BUILT-UP COLUMNS



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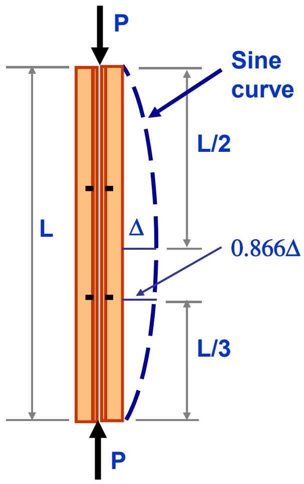
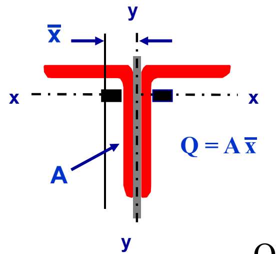
COMPRESSION MEMBERS
BUILT-UP COLUMNS

Shear Connector Design Requirements for Slip





$$\text{Total Shear Force} = P\Delta\frac{Q}{I_y}$$

$$\Delta = \frac{\Delta_0}{1-\frac{P}{P_{cr}}}$$

where $\Delta_0 = 0.001L$ and $P/P_{cr} = 0.877$, then

$$\text{Required Slip Resistance} = 0.008P_uL\frac{Q}{I_y}$$

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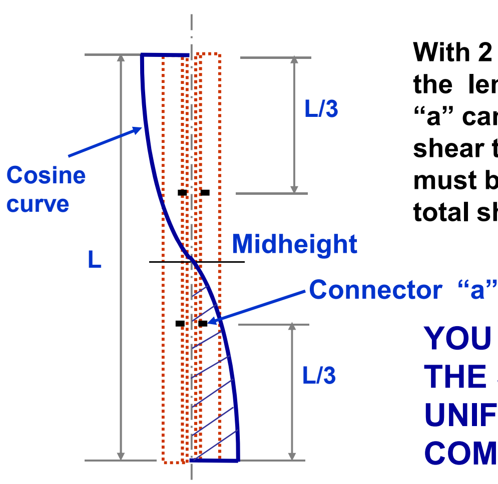
COMPRESSION MEMBERS
BUILT-UP COLUMNS

Shear Force Distribution Along Length of the Column



With 2 intermediate fasteners along the length of the column, connector "a" can only provide 13% of the shear transfer. The end connection must be designed for 87% of the total shear force.

YOU CANNOT DISTRIBUTE THE SHEAR CONNECTORS UNIFORMLY AS IN COMPOSITE BEAMS

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COMPRESSION MEMBER DESIGN EXAMPLE 5

ANSI/AISC 360-10 *Specification for the Design of Steel Buildings*, 2010

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COMPRESSION MEMBERS
BUILT-UP COLUMNS

Design Slip Resistance – RCSC Specification for Structural Joints Using ASTM A325 and A490 Bolts

Example:	**2L 5 x 3 ¼**	
P_u = 46.8k	**F_y = 36 ksi**	**L = 150 in.**
A = 3.88 in.	**I_y = 5.53 in.^{4}**	**$\bar{x}$ = 0.648 in.**
Double Angles have $^3/_8$ in. separation		

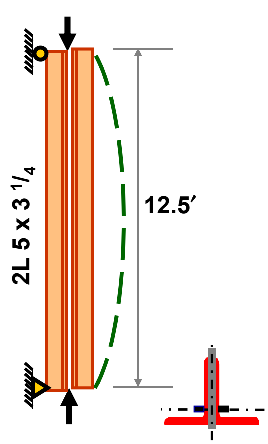
$$Q = \frac{3.88}{2}\left(0.648 + \frac{0.375}{2}\right) = 1.62\,\text{in.}^3$$

$$\text{S.F.} = 0.008(46.8)(150)\frac{1.62}{5.53} = 16.45^k$$

The shear force (single shear) is 35% of P_u

For a slip = $^1/_{32}$ in. at the ends,

the mid-height deflection will be 2 ¼ in.



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SUMMARY

- A better understanding of the stability design for various structural shapes has been provided.
- The design limit states of flexural, torsional, and flexural-torsional buckling resistance of compression members have been reviewed.
- The most frequently applied AISC LRFD and ASD design strengths for compression members provided in the Steel Specification have been discussed.
- The use of design examples and video demonstrations has helped our understanding of stability design.

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POLLING QUESTION 2

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Up Next…

Session 3: June 26 –
Behavior of Flexural Members
by R.D. Ziemian, PE, PhD

Using an approach similar to that employed in Session 1, this lecture will begin by presenting and dissecting the solution to the differential equation that defines the elastic lateral torsional buckling (LTB) strength of beams. Related flexural and torsional concepts, including the benefits of warping resistance, will be briefly reviewed. The assumption of elastic behavior will then be relaxed to define the inelastic LTB and plastic moment capacities of flexural members. The strength of beams without slender elements will be covered and ultimately presented in the form of beam resistance curves.

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Individual Webinar Registrants

CEU/PDH Certificates

Within 2 business days…

- You will receive an email on how to report attendance from: registration@aisc.org.
- Be on the lookout: Check your spam filter! Check your junk folder!
- Completely fill out online form. Don't forget to check the boxes next to each attendee's name!

Individual Webinar Registrants

CEU/PDH Certificates

Within 2 business days…

- New reporting site (URL will be provided in the forthcoming email).
- Username: Same as AISC website username.
- Password: Same as AISC website password.

8-Session Registrants

CEU/PDH Certificates

One certificate will be issued at the conclusion of all 8 sessions.

8-Session Registrants

Access to the quiz: Information for accessing the quiz will be emailed to you by Wednesday. It will contain a link to access the quiz. EMAIL COMES FROM NIGHTSCHOOL@AISC.ORG

Quiz and Attendance records: Posted Tuesday mornings. www.aisc.org/nightschool - click on Current Course Details.

Reasons for quiz:

- EEU – must take all quizzes and final to receive EEU
- CEUs/PDHS – If you watch a recorded session you must take quiz for CEUs/PDHs.
- REINFORCEMENT – Reinforce what you learned tonight. Get more out of the course.

NOTE: If you attend the live presentation, you do not have to take the quizzes to receive CEUs/PDHs.

8-Session Registrants

Access to the recording: Information for accessing the recording will be emailed to you by this Wednesday. The recording will be available for three weeks. For 8-session registrants only. EMAIL COMES FROM NIGHTSCHOOL@AISC.ORG.

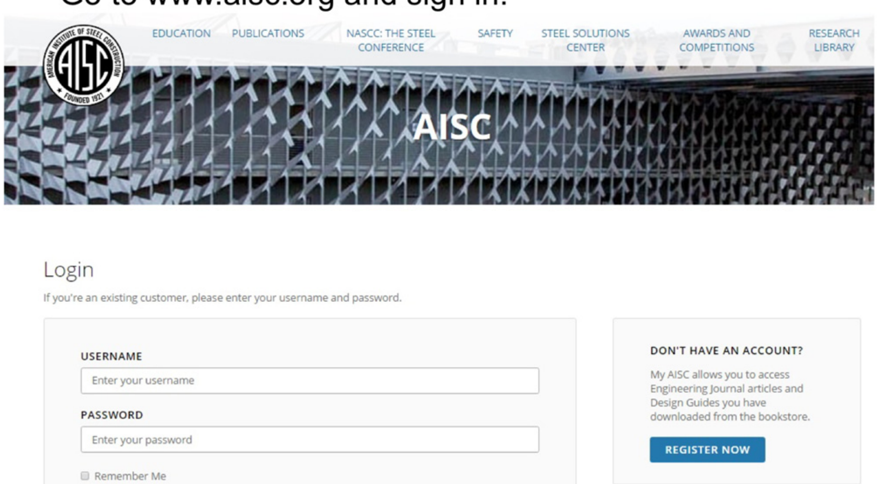
CEUs/PDHS – If you watch a recorded session you must take AND PASS the quiz for CEUs/PDHs.

Night School Resources for 8-session package Registrants

Find all your handouts, quizzes and quiz scores, recording access, and attendance information all in one place!

Night School Resources for 8-session package Registrants

Go to www.aisc.org and sign in.



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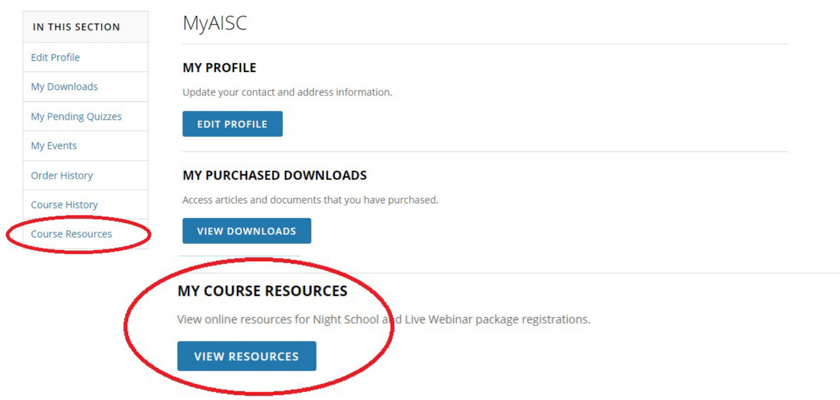
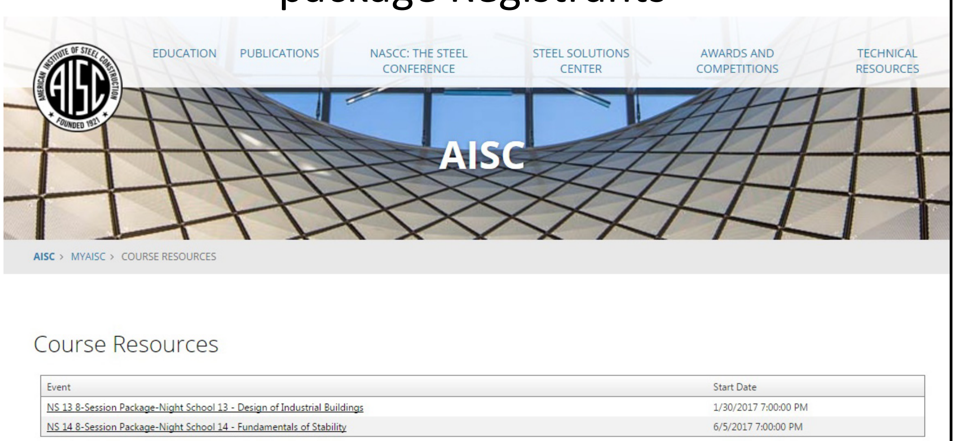


Night School Resources for 8-session package Registrants



Night School Resources for 8-session package Registrants

Night School 13: Design of Industrial Buildings

8-SESSION PACKAGE RESOURCES

Event	Date	Handouts	Video	Quiz	Attendance
NS13 - Design Criteria	1/30/2017 7:00:00 PM	Handouts	View Passcode: NS13DSN	Pass Score: 80	Pending
NS13 - Economic Considerations	2/6/2017 7:00:00 PM	Handouts	Available 02/08/2017 5pm EST	Available 02/08/2017 5pm EST	Pending
NS13 - Lateral Load Systems and Details	2/13/2017 7:00:00 PM	Handouts	Available 02/15/2017 5pm EST	Available 02/15/2017 5pm EST	Pending
NS13 - Preliminary Design Procedures	2/27/2017 7:00:00 PM	Handouts	Available 03/01/2017 5pm EST	Available 03/01/2017 5pm EST	Pending
NS13 - Crane Girder Design and Frame Analysis	3/6/2017 7:00:00 PM	Handouts	Available 03/08/2017 5pm EST	Available 03/08/2017 5pm EST	Pending
NS13 - Frame Member and Connection Design	3/13/2017 7:00:00 PM	Handouts	Available 03/15/2017 5pm EST	Available 03/15/2017 5pm EST	Pending
NS13 - Transfer Crane Girder & Longitudinal Bldg Bracing Dsn	3/27/2017 7:00:00 PM	Handouts	Available 03/29/2017 5pm EST	Available 03/29/2017 5pm EST	Pending
NS13 - Building Envelope and Bracing Design	4/3/2017 7:00:00 PM	Handouts	Available 04/05/2017 5pm EST	Available 04/05/2017 5pm EST	Pending
NS13 - Final Exam	4/10/2017 7:00:00 PM			Available 04/12/2017 5pm EST	

Night School Resources for 8-session package Registrants

- Weekly “quiz and recording” email.
- Weekly updates of the master Quiz and Attendance record found at www.aisc.org/nightschool. Scroll down to Quiz and Attendance records.
  - Updated on Tuesday mornings.

Night School Resources for 8-session package Registrants

- Webinar connection information:
  - Found in your registration confirmation/receipt.
  - Reminder email sent out Monday mornings.
- Link to handouts also found here.

Thank You

There's always a solution in steel.

Please give us your feedback!
Survey at conclusion of webinar.

structural STEEL

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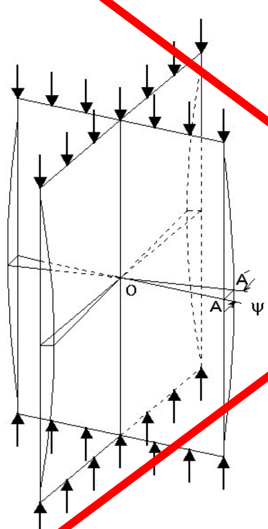
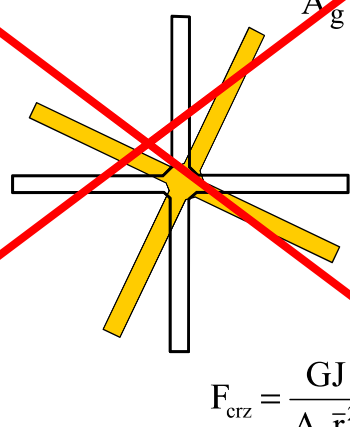
TORSIONAL BUCKLING
CRUCIFORM MEMBERS



$$\bar{r}_o^2 = x_o^2 + y_o^2 + \frac{I_x + I_y}{A_g} \qquad \text{(E4 - 11)}$$



$$F_{crz} = \frac{GJ}{A_g \bar{r}_o^2} \qquad \text{(E4 - 3)}$$

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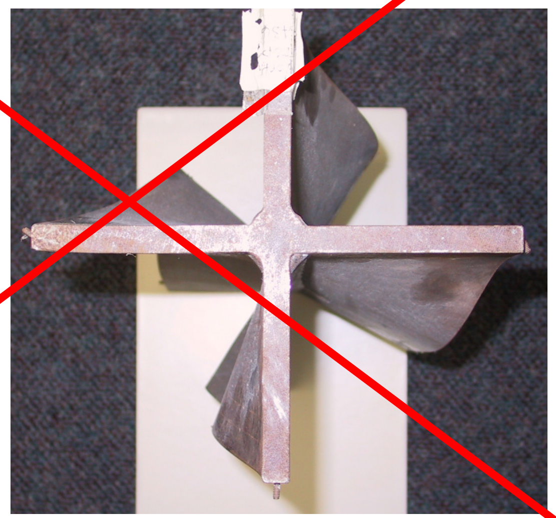
TORSIONAL BUCKLING
CRUCIFORM MEMBERS

Twist about the shear center of the section.



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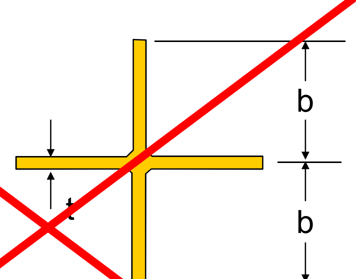
TORSIONAL BUCKLING
CRUCIFORM MEMBERS

$$J = \frac{4bt^3}{3}$$

$$I_x = I_y = 2\frac{b^3 t}{3}$$



$$F_{crz} = \frac{GJ}{A\bar{r}_o^2} = \frac{GJ}{I_x + I_y} = \frac{G}{\left(\frac{b}{t}\right)^2} = \frac{0.385E}{\left(\frac{b}{t}\right)^2}$$

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DESIGN STRENGTH
SINGLE ANGLE COLUMN ∟2 x 2 x 3/16

Buckling about z-axis

$$\left(\frac{KL}{r}\right)_z = \frac{44}{0.389} = 113 \leq 4.71\sqrt{\frac{E}{F_y}} = 133 \qquad F_y = 36\,\text{ksi}$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = 22.41\,\text{ksi}$$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y = 18.38\,\text{ksi}$$

$$P_n = F_{cr} A_g = 13.27\,\text{ksi}$$

$$\text{ASD} \quad \frac{P_n}{\Omega} = 7.95\,\text{kips} \qquad \text{LRFD} \quad \phi P_n = 11.94\,\text{kips}$$

Buckling about z-axis does not control

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COMPRESSION MEMBER DESIGN EXAMPLE 4

ANSI/AISC 360-10 *Specification for the Design of Steel Buildings*, 2010

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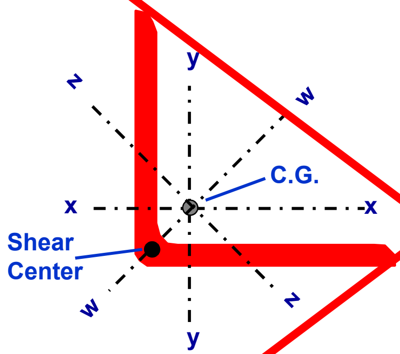
MEMBER DESIGN STRENGTH
SINGLE ANGLE COLUMN ∟6 x 6 x 5/8



Section Properties:

A_g = 7.13 in.^{2}
$I_x = I_y$ = 24.1 in.^{4}
J = 0.955 in.^{4}
C_w = 2.5 in.^{6}
r_x = 1.84 in.
r_z = 1.17 in.
$\bar{r}_o$ = 3.28 in.
Q= 1.0
H = 0.631

Material Properties:

F_y = 50 ksi
E = 29,000 ksi
G = 0.385 E

Length: KL = L = 10 ft.

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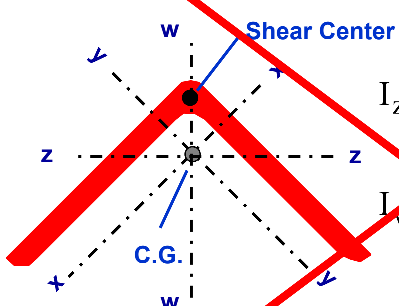
MEMBER DESIGN STRENGTH
SINGLE ANGLE COLUMN ∟6 x 6 x 5/8

Flexural-Torsional Buckling



$$I_z = A_g r_z^2 = (7.13)(1.17)^2 = 9.76 \text{ in.}^2$$

$$I_w = I_x + I_y - I_z$$
$$= 2(24.1) - 9.76 = 38.44 \text{ in.}^4$$

$$r_w = \sqrt{\frac{I_w}{A_g}} = \sqrt{\frac{38.44}{7.13}} = 2.32 \text{ in.}$$

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MEMBER DESIGN STRENGTH

SINGLE ANGLE COLUMN ∟6 x 6 x 5/8

Flexural-Torsional Buckling

$$F_{ez} = \left(\frac{\pi^2 E C_w}{(K_z L)^2} + GJ \right) \frac{1}{A_g \bar{r}_o^2} \cong \frac{GJ}{A_g \bar{r}_o^2}$$

Small compared to GJ, therefore this term can be safely ignored for angles, double angles and tees
In this case, 50 versus 10700

$$F_{ez} = \frac{GJ}{A_g \bar{r}_o^2} = \frac{(11200)(0.955)}{(7.13)(3.28)^2} = 139.44\,\text{ksi}$$

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MEMBER DESIGN STRENGTH

SINGLE ANGLE COLUMN ∟6 x 6 x 5/8

Flexural-Torsional Buckling

$$F_{ew} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)_w^2} = 106.98\,\text{ksi}$$

$$F_e = \frac{F_{ew} + F_{ez}}{2H}\left[1 - \sqrt{1 - \frac{4F_{ew}F_{ez}H}{(F_{ew} + F_{ez})^2}}\right] \qquad \text{(E4 - 5)}$$

$$F_e = \frac{106.98 + 139.44}{2(0.631)}\left[1 - \sqrt{1 - \frac{4(106.98)(139.44)(0.631)}{(106.98 + 139.44)^2}}\right]$$

$$F_e = 74.90\,\text{ksi}$$

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DESIGN STRENGTH
SINGLE ANGLE COLUMN ∟6 x 6 x 5/8

Buckling about z-axis

$$\left(\frac{KL}{r}\right)_z = \frac{(12)(10)}{1.17} = 103 \le 4.71\sqrt{\frac{E}{F_y}} = 113 \quad \text{or}$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)_z^2} = 27.21\,\text{ksi} \qquad \frac{F_y}{F_{ez}} = \frac{50}{27.21} = 1.837$$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y = 23.18\,\text{ksi} \quad \text{(E3 - 2)}$$

$$P_n = F_{cr}A_g = 165.25\,\text{kips} \quad \text{(E3 - 1)}$$

$$\text{ASD} \quad \frac{P_n}{\Omega} = 98.95\,\text{kips} \qquad \text{LRFD} \quad \phi P_n = 148.72\,\text{kips}$$

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MEMBER DESIGN STRENGTH
SINGLE ANGLE COLUMN ∟6 x 6 x 5/8

Flexural-Torsional Buckling

$$\left(\frac{KL}{r}\right)_{F-T} = \frac{(12)(10)}{2.32} = 52 \le 4.71\sqrt{\frac{E}{F_y}} = 113 \quad \text{or}$$

$$\frac{F_y}{F_{eF-T}} = \frac{50}{74.90} = 0.667$$

$$F_{crF-T} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y = 37.82\,\text{ksi}$$

$$P_n = F_{cr}A_g = 269.66\,\text{kips}$$

$$\text{ASD} \quad \frac{P_n}{\Omega} = 161.47\,\text{kips} \qquad \text{LRFD} \quad \phi P_n = 242.69\,\text{kips}$$

Buckling about z-axis controls

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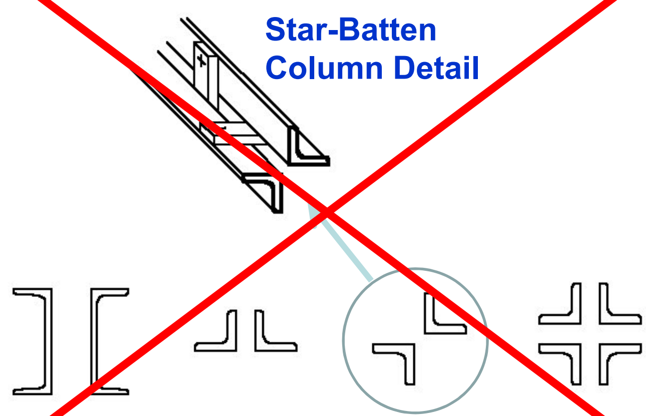
COMPRESSION MEMBERS
BUILT-UP COLUMNS



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POLLING QUESTION 1

The AISC 360-10 Specification defines which buckling limit states for compression members?

- A. Flexural and Local Plate
- B. Torsional and Flexural-Torsional
- C. Flexural and Distortional

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