

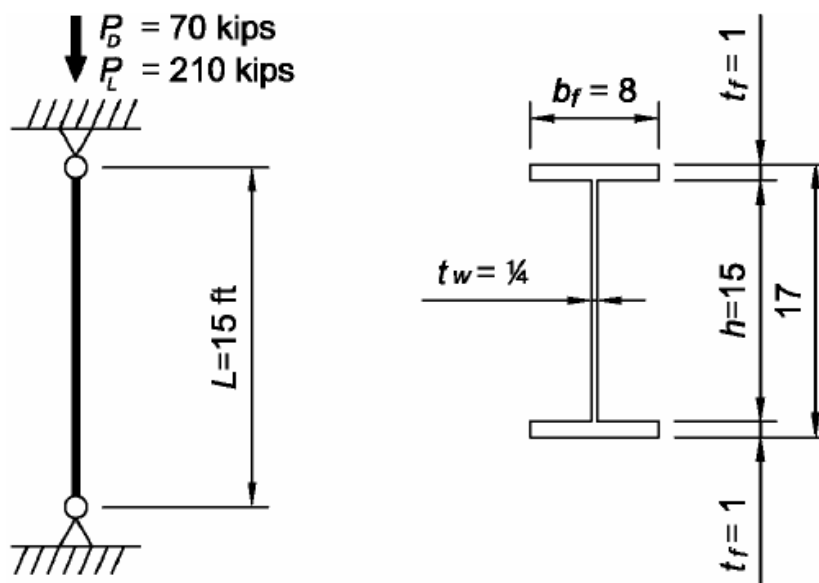
AISC 360-05 Example 002

BUILT UP WIDE FLANGE MEMBER UNDER COMPRESSION

EXAMPLE DESCRIPTION

A demand capacity ratio is calculated for the built-up, ASTM A572 grade 50, column shown below. An axial load of 70 kips (D) and 210 kips (L) is applied to a simply supported column with a height of 15 ft.

GEOMETRY, PROPERTIES AND LOADING



TECHNICAL FEATURES TESTED

- Section compactness check (compression)
- Warping constant calculation, C_w
- Member compression capacity with slenderness reduction

PROGRAM NAME: SAP2000
REVISION NO.: 0

RESULTS COMPARISON

Independent results are hand calculated and compared with the results from Example E.2 AISC *Design Examples, Volume 13.0* on the application of the 2005 AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360-05).

Output Parameter	SAP2000	Independent	Percent Difference
Compactness	Slender	Slender	0.00%
$\phi_c P_n$ (kips)	506.1	506.1	0.00 %

COMPUTER FILE: AISC 360-05 Ex002

CONCLUSION

The results show an exact comparison with the independent results.

HAND CALCULATION

Properties:

Material: ASTM A572 Grade 50

$$E = 29,000 \text{ ksi}, F_y = 50 \text{ ksi}$$

Section: Built-Up Wide Flange

$$d = 17.0 \text{ in}, b_f = 8.00 \text{ in}, t_f = 1.00 \text{ in}, h = 15.0 \text{ in}, t_w = 0.250 \text{ in}.$$

Ignoring fillet welds:

$$A = 2(8.00)(1.00) + (15.0)(0.250) = 19.75 \text{ in}^2$$

$$I_y = \frac{2(1.0)(8.0)^3}{12} + \frac{(15.0)(0.25)^3}{12} = 85.35 \text{ in}^3$$

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{85.4}{19.8}} = 2.08 \text{ in}.$$

$$I_x = \sum A d^2 + \sum I_x$$

$$I_x = 2(8.0)(8.0)^2 + \frac{(0.250)(15.0)^3}{12} + \frac{2(8.0)(1.0)^3}{12} = 1095.65 \text{ in}^4$$

$$d' = d - \frac{t_1 + t_2}{2} = 17 - \frac{1+1}{2} = 16 \text{ in}$$

$$C_w = \frac{I_y \cdot d'^2}{4} = \frac{(85.35)(16.0)^2}{4} = 5462.583 \text{ in}^4$$

$$J = \sum \frac{b t^3}{3} = \frac{2(8.0)(1.0)^3 + (15.0)(0.250)^3}{3} = 5.41 \text{ in}^4$$

Member:

$K = 1.0$ for a pinned-pinned condition

$L = 15 \text{ ft}$

Loadings:

$$P_u = 1.2(70.0) + 1.6(210) = 420 \text{ kips}$$

Section Compactness:

Check for slender elements using Specification Section E7

Localized Buckling for Flange:

$$\lambda = \frac{b}{t} = \frac{4.0}{1.0} = 4.0$$

$$\lambda_p = 0.38 \sqrt{\frac{E}{F_y}} = 0.38 \sqrt{\frac{29000}{50}} = 9.152$$

$$\lambda < \lambda_p, \text{ No localized flange buckling}$$

Flange is Compact.

Localized Buckling for Web:

$$\lambda = \frac{h}{t} = \frac{15.0}{0.250} = 60.0,$$

$$\lambda_r = 1.49 \sqrt{\frac{E}{F_y}} = 1.49 \sqrt{\frac{29000}{50}} = 35.9$$

$$\lambda > \lambda_r, \text{ Localized web buckling}$$

Web is Slender.

Section is Slender

Member Compression Capacity:

Elastic Flexural Buckling Stress

Since the unbraced length is the same for both axes, the y-y axis will govern by inspection.

$$\frac{KL_y}{r_y} = \frac{1.0(15 \cdot 12)}{2.08} = 86.6$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 \cdot 29000}{(86.6)^2} = 38.18 \text{ ksi}$$

Elastic Critical Torsional Buckling Stress

Note: Torsional buckling will not govern if $KL_y > KL_z$, however, the check is included here to illustrate the calculation.

$$F_e = \left[\frac{\pi^2 EC_w}{(K_z L)^2} + GJ \right] \frac{1}{I_x + I_y}$$

$$F_e = \left[\frac{\pi^2 \cdot 29000 \cdot 5462.4}{(180)^2} + 11200 \cdot 5.41 \right] \frac{1}{1100 + 85.4} = 91.8 \text{ ksi} > 38.18 \text{ ksi}$$

Therefore, the flexural buckling limit state controls.

$$F_e = 38.18 \text{ ksi}$$

Section Reduction Factors

Since the flange is not slender,

$$Q_s = 1.0$$

Since the web is slender,

For equation E7-17, take f as F_{cr} with $Q = 1.0$

$$4.71 \sqrt{\frac{E}{QF_y}} = 4.71 \sqrt{\frac{29000}{1.0(50)}} = 113 > \frac{KL_y}{r_y} = 86.6$$

So

$$f = F_{cr} = Q \left[0.658^{\frac{QF_y}{F_e}} \right] F_y = 1.0 \left[0.658^{\frac{1.0(50)}{38.2}} \right] \cdot 50 = 28.9 \text{ ksi}$$

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

$$b_e = 1.92(0.250) \sqrt{\frac{29000}{28.9}} \left[1 - \frac{0.34}{(15.0/0.250)} \sqrt{\frac{29000}{28.9}} \right] \leq 15.0 \text{ in}$$

$$b_e = 12.5 \text{ in} \leq 15.0 \text{ in}$$

therefore compute A_{eff} with reduced effective web width.

$$A_{eff} = b_e t_w + 2b_f t_f = (12.5)(0.250) + 2(8.0)(1.0) = 19.1 \text{ in}^2$$

where A_{eff} is effective area based on the reduced effective width of the web, b_e .

$$Q_a = \frac{A_{eff}}{A} = \frac{19.1}{19.75} = 0.968$$

$$Q = Q_s Q_a = (1.00)(0.968) = 0.968$$

Critical Buckling Stress

Determine whether Specification Equation E7-2 or E7-3 applies

$$4.71 \sqrt{\frac{E}{QF_y}} = 4.71 \sqrt{\frac{29000}{0.966(50)}} = 115.4 > \frac{KL_y}{r_y} = 86.6$$

Therefore, Specification Equation E7-2 applies.

When $4.71 \sqrt{\frac{E}{QF_y}} \geq \frac{KL}{r}$

$$F_{cr} = Q \left[0.658^{\frac{QF_y}{F_e}} \right] F_y = 0.966 \left[0.658^{\frac{1.0(50)}{38.18}} \right] \bullet 50 = 28.47 \text{ ksi}$$

Nominal Compressive Strength

$$P_n = F_{cr} A_g = 28.5 \bullet 19.75 = 562.3 \text{ kips}$$

$$\phi_c = 0.90$$

$$\phi_c P_n = F_{cr} A_g = 0.90(562.3) = 506.1 \text{ kips} > 420 \text{ kips}$$

$$\phi_c P_n = 506.1 \text{ kips}$$